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TRANSACTIONS

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Page 255, Equation (22), for

$$\text{“ } M_0 = \frac{2\,p\,k^2}{6}\,\frac{1-\frac{x^2}{6}}{-\frac{x^2}{18}+4\,\frac{k^2}{L^2}\left(2-\frac{4}{3}\,x^2+\frac{5}{18}\,x^4-\frac{x^6}{54}\right)}\text{ ”, read}$$

$$\text{“ } M_0 = -\frac{p\,k^2}{3}\,\frac{\left(1-\frac{13}{60}\,x^2+\frac{5}{280}\,x^4-\ldots\right)}{\frac{2}{45}\,x^2+\ldots+8\,\frac{k^2}{L^2}\left(1-\frac{2}{3}\,x^2+\frac{2}{9}\,x^4+\ldots\right)}\text{ ”}$$

Page 255, Equation (23), for

$$\text{“ } M = \frac{2\,p\,k^2}{6}\,\frac{2-\frac{7}{12}\,x^2+\frac{x^4}{24}}{-\frac{x^2}{18}+4\,\frac{k^2}{L^2}\left(2-\frac{4}{3}\,x^2+\frac{5}{18}\,x^4-\frac{x^6}{54}\right)}\text{ ”, read}$$

$$\text{“ } M_1 = \frac{p\,k^2}{3}\,\frac{\left(2-\frac{8}{15}\,x^2+\frac{2}{35}\,x^4-\ldots.\right)}{\frac{2}{45}\,x^2+\ldots.+8\,\frac{k^2}{L^2}\left(1-\frac{2}{3}\,x^2+\frac{2}{9}\,x^4+\ldots.\right)}\text{ ”}$$

Page 469, line 25, for “It is essential that streams and lakes be protected against pollution, as in the future, all large public water supplies are, or must be, derived from surface sources”, read “It is essential that streams and lakes be protected against pollution as all large public water supplies are, or in the future must be, derived from surface sources”.

Page 1141, line 4 from bottom, strike out the words, “as to”.

Page 1142, line 17, insert the word “reliable” after the word “less”, making the sentence read “The tests quoted by Mr. Wegmann * * * vary so greatly that even the mean value is less reliable than the former tests.”

Page 1142, line 3 from bottom, for “it is carried along” read “carries it along”.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 1509

ENGINEERING GEOLOGY OF THE CATSKILL WATER SUPPLY*

By Charles P. Berkey,† Esq., and James F. Sanborn,‡ M. Am. Soc. C. E.

With Discussion by Messrs. Alfred D. Flinn, George G. Honness, Lazarus White, Walter E. Spear, Arthur S. Tuttle, Louis L. Tribus, E. G. Haines, X. Henry Goodnough, William W. Brush, Thomas H. Wiggin, Joel D. Justin, James F. Kemp, Thaddeus Merriman, and Charles P. Berkey and James F. Sanborn.

Synopsis

This paper deals with the geological features involved in the location, design, and construction of the Catskill Water Supply System of the City of New York.

The purpose is to discuss the intimate relation of applied geology and certain phases of engineering work, to present illustrations of the method used by the engineers in solving, through the correct use of geological information, some of the problems involved, and to show the relations of these engineering problems to the natural features and conditions of the country in which the work is located.

General Conditions

In planning the construction of a water-works system comprising an aqueduct more than 100 miles long, together with several dams and other structures, the choice of location, design of structures, and details of construction were greatly influenced by the form, relief, and underground condition of the

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country traversed. Seldom in a large public enterprise of this kind, has so much trouble been taken to discover and to work out the important relations of topographic form and underground structure, and to apply the results of these studies to the problems of location, design, construction, and operation of the completed system. Indeed, it was necessary to success that all the conditions be known in detail. The selection of proper dam sites, with foundations sound enough to support the enormous weight of the dams and impervious enough to prevent leakage, demanded a careful investigation and study of all the intricacies of the physical condition and behavior of the ground. The expedient of conveying water in tunnels driven deep in bed-rock, under great hydrostatic pressure, was adopted only after sufficient information was available to give confidence in the general stability of local geological conditions.

The importance of these conditions in controlling the location of the aqueduct is well illustrated in the choice of the crossing of the Hudson River, the final selection of which involved a change in the position of the entire aqueduct line west of the river. Fig. 1 is a map of the Catskill Water Supply System. The principal types of construction included in the project are, as follows:

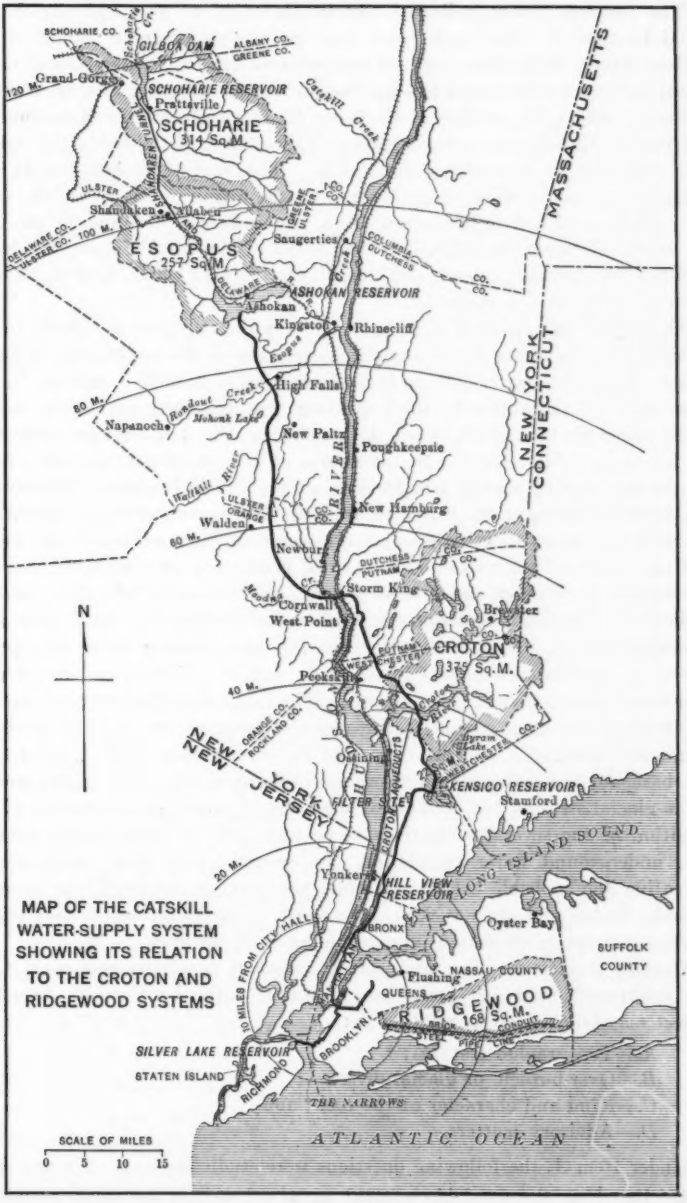
(*a*) Three high masonry dams;
(*b*) Several miles of earth dikes with core-walls; and
(*c*) An aqueduct, made up of the following parts:

Cut-and-cover aqueduct	51.0 miles
Grade tunnel (including Shandaken Tunnel)	31.7 "
Pressure tunnel	35.6 "
Pressure aqueduct	4.8 "
Steel pipe siphons	6.2 "
Pipe conduit	15.4 "
Total	144.7 miles

In the construction of pressure and grade tunnels, sixty-seven working shafts, from 150 to 1 100 ft. in depth, were sunk.

Geological Investigations

The development of the Catskill Water Supply System has been influenced by geological factors to a greater extent than is usual in most engineering enterprises. This is due in part to the great distance the water must be carried to reach New York City, the topographic and structural complexities and peculiarities of the intervening country, and the general plan of operation chosen for this particular system. Because of the type of design of the Catskill Aqueduct and the peculiar relief of the country, it was evident early in the work that many grade tunnels would be required to pierce the mountain ridges standing above hydraulic grade along any possible route. It was also found that it would be more economical, and would result in a more successful and permanent public work, if the valleys and river courses which lie much below hydraulic grade, were crossed by pressure tunnels constructed in the bed-rock.



MAP OF THE CATSKILL WATER-SUPPLY SYSTEM SHOWING ITS RELATION TO THE CROTON AND RIDGEWOOD SYSTEMS

FIG. 1.

The earliest studies indicated certain places where thorough exploration would be needed. One such place was the trough or inner valley of the Hudson River, which presented an unusual problem in the question of depth to bed-rock. To collect and to state the broad facts relating to this situation and many others, the services of Professor W. O. Crosby, of the Massachusetts Institute of Technology, and of Professor J. F. Kemp, of Columbia University, were obtained as Consulting Geologists. The studies undertaken by the geologists covered a wide range, bearing on the origin of the rocks, the processes which modified or preserved them, the influence of climatic changes, the effect of former elevation and depression of the earth's crust, the interpretation of surface features, and many other factors which tend to modify, preserve, or to destroy the rocks.

In projects of this kind it is important for the engineer to submit to the geologist certain rather definite questions relating to the engineering requirements. It is not practicable to investigate wholly unrelated matters in the early stages of the work. In the beginning, the geologist's advice was sought in directing the exploratory work, to supplement the data already secured as the boring work developed. In the course of the investigations and studies for the engineering design and location of the Catskill System, information was secured relating to the suitability of materials of construction, comparison was made of factors affecting the cost of excavation, facts relating to the water-bearing quality of the rocks and soils to be penetrated were obtained, and the influence of the rock on the structures contemplated was noted. One question studied was whether the tunnels far below the surface may be damaged by earthquake shock, or are otherwise unstable, and whether there are special sources of trouble or weakness in the rock, such as acid-bearing waters with their own peculiar destructive influence. The range of information ultimately required covered such special features as a comprehensive regional study of pre-glacial topography and drainage, and the distribution of the glacial drift as a basis for indicating the location and probable depth of the buried gorges of pre-glacial streams, the contour of the bed-rock surface, a discussion of the condition of the several formations to be met, their behavior under excavation, underground water circulation, the occurrence of zones of weakness indicating the probable extent of tunnel roof support required, the location of folds, faults, and other rock features, with a consideration of their effects on the construction operations and permanence of the work.

General Questions.—The engineering questions involving geological facts and interpretation which arose in connection with the Catskill Aqueduct project may be grouped as relating to:

A.—Topographic features;
B.—Over-burden, or glacial drift;
C.—Kind and character of the rock; and
D.—Advisory matters.

Under Item *A*, the following questions were studied:

1.—Based on the available contour maps, where are the most promising sites for the particular structures under consideration, for example, the main dams for storage reservoirs?

2.—Do the surface features indicate whether the relief is controlled by the glacial drift or by erosion of bed-rock?

3.—Considering the larger topographic features, such as the main valleys and their origin, is there any likelihood of finding deep buried gorges requiring extensive exploration?

Under Item *B*, the following questions arise:

1.—What is the thickness and distribution of the over-burden, or drift, in the area involved?

2.—Is there unusually deep cover at any particular place, or a pre-glacial gorge of uncertain depth?

3.—What is the character of the over-burden? Is it residuary soil, stream deposit, or glacial drift? Is it impervious or very porous? Will it be firm or cave in excavation? Are extensive exploratory borings or test-pits required?

Under Item *C*, some of the questions are:

1.—What is the probable rock-floor contour or profile? What is the rock formation or kind of rock forming the floor, its quality, and structural character? Is it porous or reasonably impervious?

2.—Is there danger of weathering, disintegrating, or swelling of the rock in tunnel excavation or open cuts? Is there likely to be considerable depth of decayed or weathered rock on top of the ledge proper?

3.—Is the rock massive or bedded? If sedimentary, what is the structural attitude? Is it faulted and folded? If the rock formation at a particular location or depth is questionable, is there a possibility of finding more satisfactory conditions either by a shift of line to a new location, or a change in depth?

Under Item *D*, the following questions may arise:

1.—Where should the exploratory work be located and the borings placed, and to what depth?

2.—Is the proposed project feasible from a geological standpoint, or are the natural difficulties insurmountable?

3.—Are there any special precautions to be taken at certain points during construction? What special points must be covered in the contracts?

4.—Is the locality capable of producing suitable stone for use in construction?

5.—Is there any likelihood of the disturbance of local conditions by the work of construction, such, for example, as change of water level, interference with local water supply, or weakening of foundations? Are the geological conditions such as to expect claims for either temporary or permanent damages due to construction?

The most economical method of exploration as far as it can be stated in general terms is to base it first on a thorough geological study, to check and interpret all data as gathered, and modify, if necessary, any additional exploratory work in accordance with the results already secured. A working hypothesis is essential. The engineers found it helpful to obtain the advice of a geologist in the interpretation of the exploratory borings as these were

made, so that the next hole could be located at the spot most likely to give further results in the light of the existing knowledge. By this method, much time was saved and the borings were made to give useful information. The methods of securing underground data included, in addition to surveys and mapping, the use of (1) test-pits, shafts, and trenches; (2) tunneling; (3) borings of various kinds; (4) pumping and pressure tests of drill holes; (5) laboratory tests, and microscopic examination; and (6) chemical analysis.

Geology of the Region Traversed by the Catskill Aqueduct

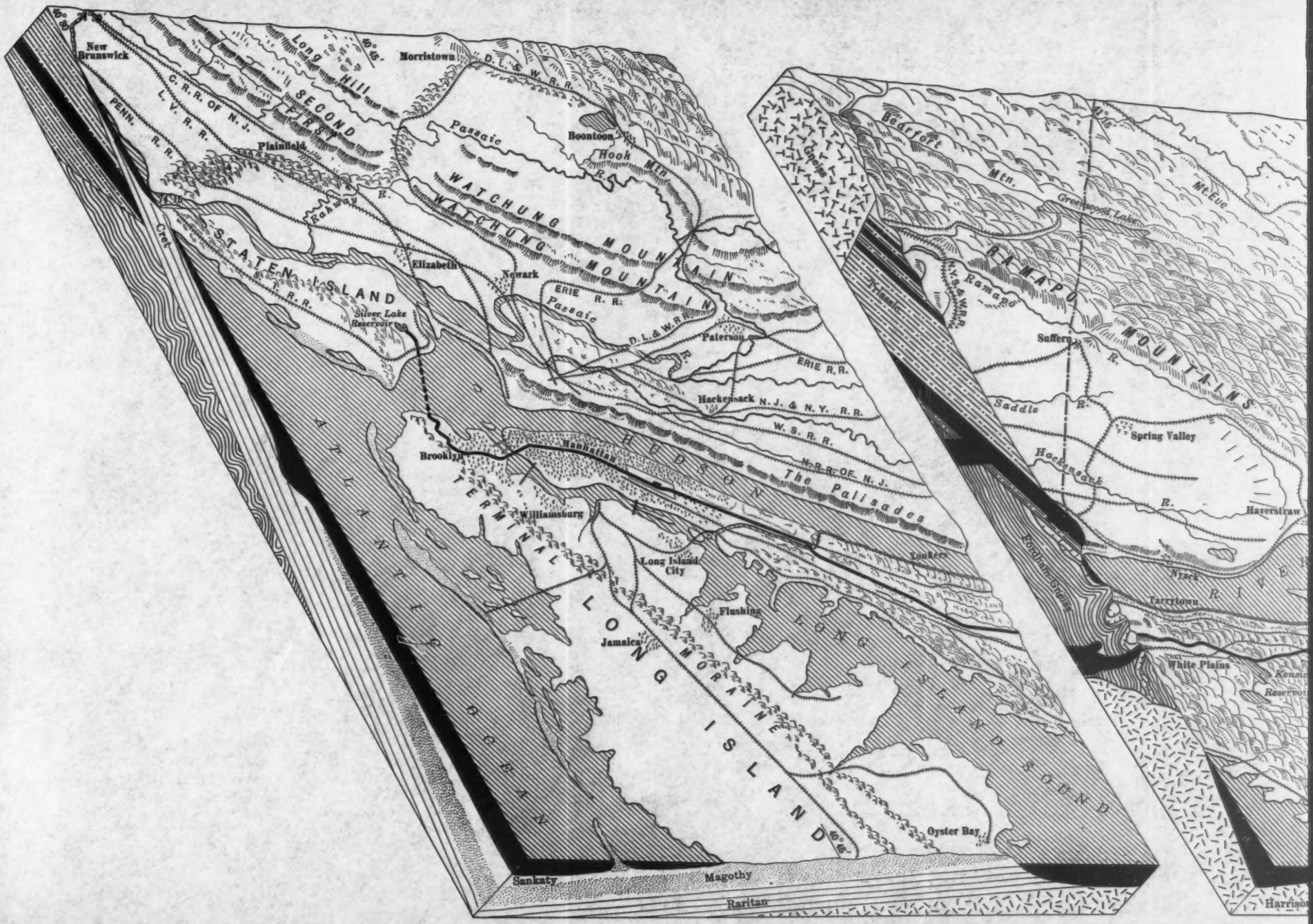
The relief features and geologic structure of the country traversed by the Catskill Aqueduct is more complex than is generally the case in such a limited area. This region has been affected by mountain foldings and accompanying dynamic disturbances at least three times. The area extends from the undisturbed interior plateau type of country (as represented by the Catskill Mountains), across the entire belt of Appalachian folds (as represented in the Middle Hudson Valley of New York), and the still earlier Taconic deformation belt (as represented in the older sedimentary rocks, especially the Hudson River slates north of the Highlands), the mountainous belt of very ancient geological formations (as represented by the Highlands with the Piedmont belt of crystalline rocks), to the Coastal Plain, including the newer formations along the present seashore. It is a complex region of a long and involved history. Fig. 2 shows the geologic column, representing the succession and variety of rocks encountered in the Catskill Water Supply development. Plate I shows a block diagram of the physiographic features, together with the underground geologic structure of the Lower Hudson region and the course of the Catskill Aqueduct.

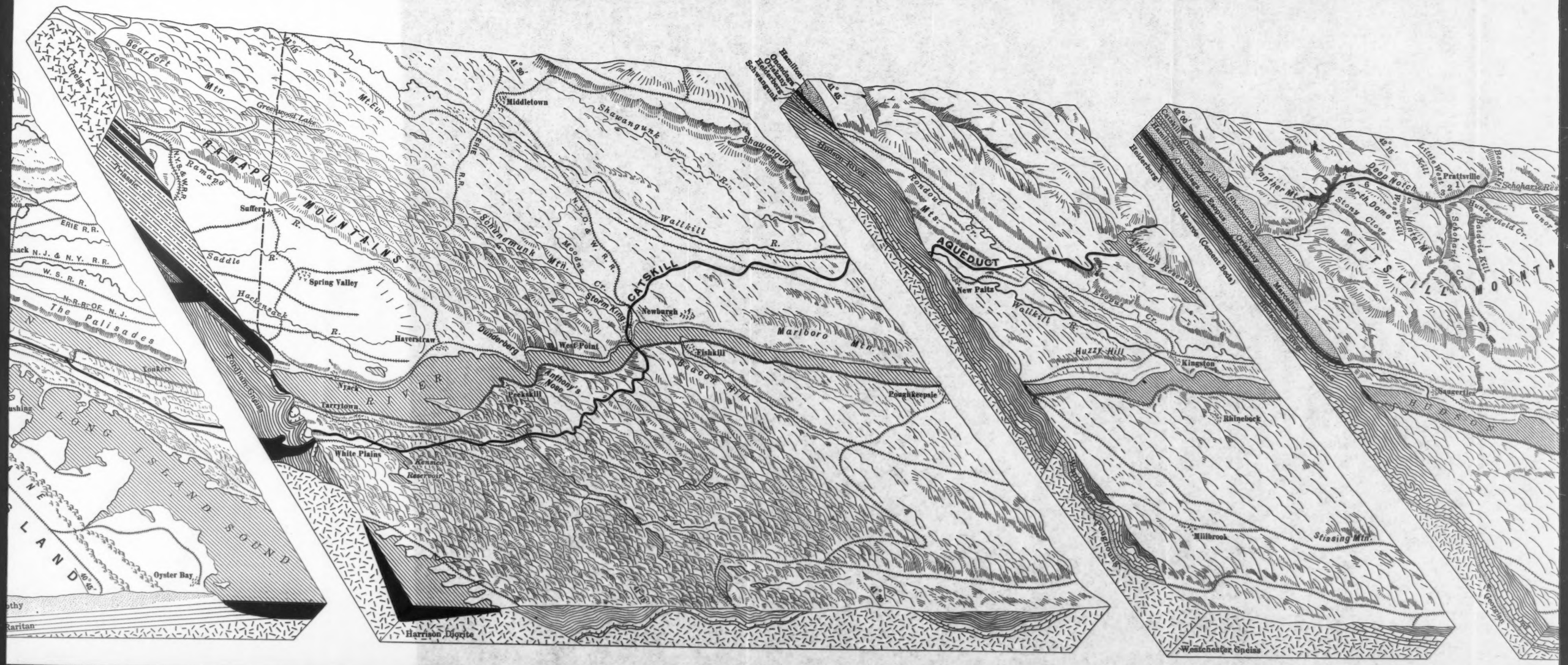
Beginning with the areas occupied by the water-sheds, the chief types of country are, as follows:

The Catskill Plateau, characterized by an elevated upland of little disturbed, rather flat-lying sediments, dipping gently westward, made up mostly of fine-grained sandstones and still finer shales.

South of the Catskill area is the Appalachian Province forming a belt running northeast-southwest, paralleling the eastern edge of the Continent. This belt has shared in the folding and other movements which have been responsible for the structural complexity of the strata and have controlled the relief habit of the present surface. A part of this belt, now somewhat lower and apparently less ridged than the remainder, forms what is known as the Great Valley. The compressive or mountain-building forces which produced the Appalachian belt, acted in a general northwesterly direction and produced folds of remarkable continuity, accompanied by a series of great faults running parallel with the main ridges and valleys, or generally northeast and southwest. The whole province is crossed by the Catskill Aqueduct.

Still farther south the country changes again, and one finds there the ancient Archean land mass, represented to-day by the gneisses and granites of the Highlands. In this belt, all the rocks are crystalline. They belong to an extremely old series, possibly the oldest known in the eastern part of the United States. Doubtless, originally many of them were sediments similar to those


New Brunswick
Morristown
Long Hill
SECOND R.
Passaic
Boontoon
Hook Mtn.
R.
WATCHUNG MOUNTAIN
WATCHUNG MOUNTAIN
Plainfield
Railway
R.
Elizabeth
Newark
ERIE R. R.
Passaic
R.
D. L. & W. R.
Paterson
ERIE R. R.
Hackensack
N. J. & N. Y. R. R.
W. S. R. R.
N. R. R. OF N. J.
The Palisades
Manhattan
HUDSON
TERMINAL
Brooklyn
Williamsburg
Long Island City
Flushing
Jamaica
LONG ISLAND
MORAINE
LONG ISLAND SOUND
Oyster Bay
Sankaty
Magothy
Raritan
ATLANTIC OCEAN
STATEN ISLAND
Silver Lake Reservoir
PENN. R. R.
L. V. R. R.
C. R. R. OF N. J.
Cret
Bedford
Mts.
Mt. Eve
Greenwood Lake
RAMAPO MOUNTAINS
Ramapo
Saffern
R.
Saddle
R.
Spring Valley
Hackensack
R.
Haverstraw
Nyack
Tarrytown
White Plains
Ponthorne Sound
Harrison


Bedford
Mt. Eve
Greenwood Lake
Greig
Middletown
Shawangunk
Shawangunk R.
Wallkill R.
Schunemunk Mtn.
Moodna Cr.
Storm King
Newburgh
CATSKILL
AQUEDUCT
Hamilton
Onondaga
Oriskany
Helderberg
Schawangunk
Hudson River
Rondout Cr.
Rondout Mts.
Esopus
Ulster
Orange
Rockland
Putnam
Westchester
Dutchess
Columbia
Greene
Albany
Schoharie
Delaware
Sullivan
Ulster
Un. Monroe (Cement Beds)
Esopus (Shenbourne)
Oriskany
Helderberg
Kingston
Rhinebeck
Hillbrook
Stissing Mtn.
Poughkeepsie
Fishkill
Beacon Hill
Mariboro Mtn.
Huzzy Hill
New Paltz
Wallkill R.
Ashokan Reservoir
Esopus Cr.
Panther Mt.
Deep Notch
Little West Kill
Prattsville
North Dome
Stony Clove
West Kill
Hunter
Batavia Kill
Hunterfield Cr.
Schoharie Cr.
Manor
Saugerties
Catskill
Hudson
Greenport

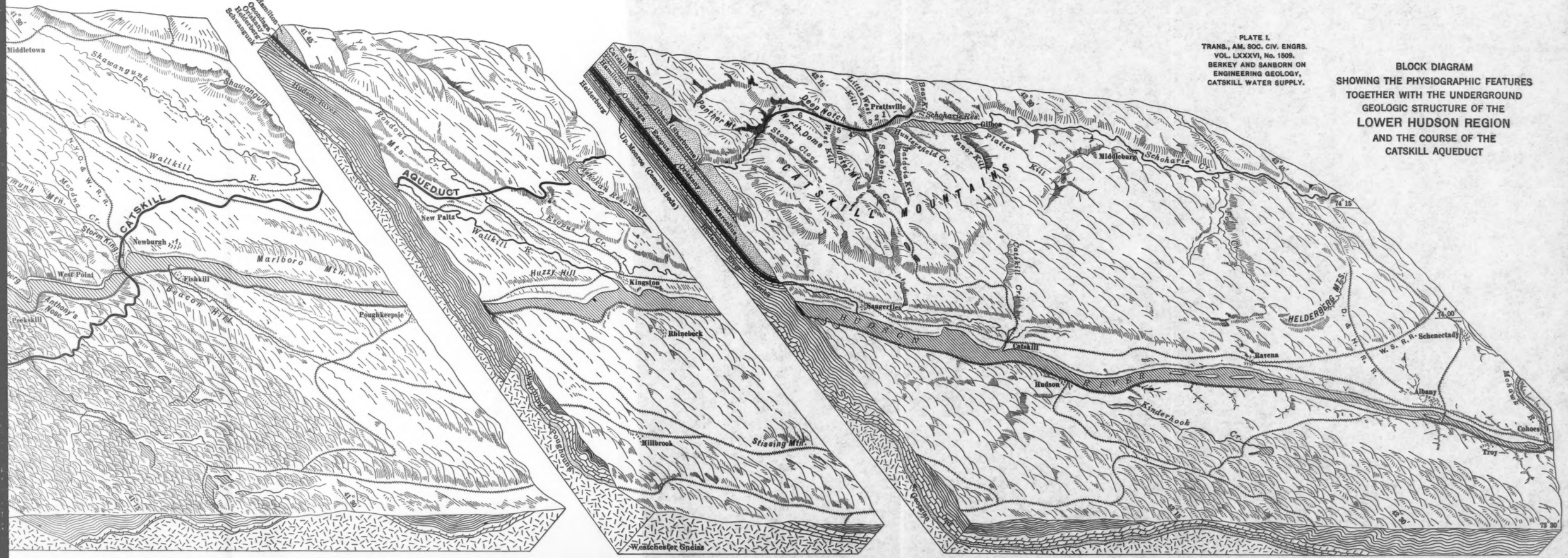
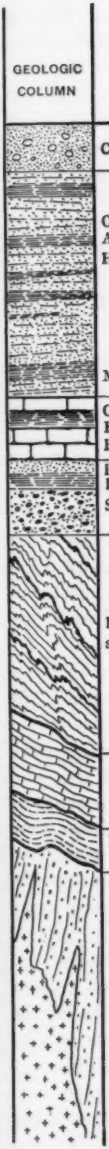


PLATE I.
TRANS., AM. SOC. CIV. ENGRS.
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BERKEY AND SANBORN ON
ENGINEERING GEOLOGY,
CATSKILL WATER SUPPLY.

BLOCK DIAGRAM SHOWING THE PHYSIOGRAPHIC FEATURES TOGETHER WITH THE UNDERGROUND GEOLOGIC STRUCTURE OF THE LOWER HUDSON REGION AND THE COURSE OF THE CATSKILL AQUEDUCT





NAME OF FORMATION	THICKNESS IN FEET	AGE	LOCALITY
Catskill Conglomerate	300	Devonian	Catskill Mountains
Oneonta Sandstones and Shales Ashokan Flags Hamilton Flags Marcellus Shale	3 000±		
Onondaga Limestone Esopus Shale Helderberg Limestones	350±		Rondout Valley
Binnewater Sandstone High Falls Shale	120	Silurian	
Shawangunk Grit Unconformity	300±		Shawangunk Mountains
Hudson River Shales, slates and graywackes	3 000 — 5 000+	Ordovician	Hudson and Wallkill Valleys
Wappinger Limestone	1 000	Cambro-ordovician	Hudson Valley
Pouquag Quartzite Unconformity	600	Cambrian	
Archaean Gneisses and Granites Manhattan Schist Inwood Limestone Fordham Gneiss Yonkers Gneiss	Unknown Great	Pre Cambrian Complex Rocks Crystalline and Metamorphic	Highlands of the Hudson, Westchester County, New York City.

FIG. 2.

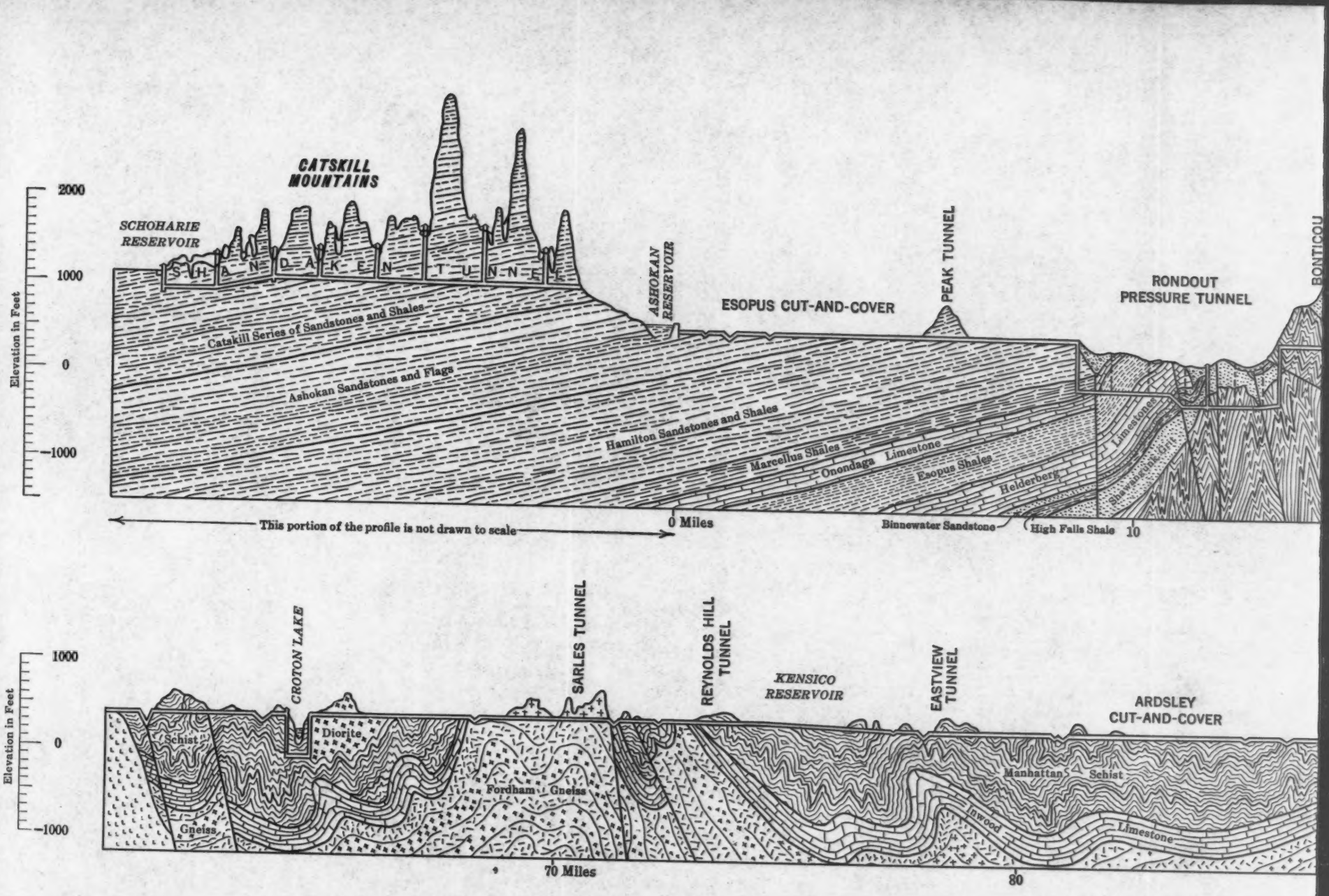
occupying the districts farther north, already described; but, subsequently, they were affected by metamorphism and invaded by igneous intrusions of many kinds, especially by granites, and thus became not only much modified from their original condition, but also much disturbed and distorted. In their present condition, they form the most complex belt with which this work has had to deal.

Although the Highlands proper is a narrow belt not more than 20 miles wide, the same kinds of rocks to a large degree extend continuously on the east side of the Hudson River to New York City. For 50 miles, the aqueduct has been located through and over these complex rocks. The southerly part of this unit has more moderate relief than the Highlands proper, and its surface gradually becomes lower and lower, passing beneath sea level in the vicinity of Long Island Sound and New York Bay, only the Highlands Belt being mountainous.

The last of the distinctly different districts follows the immediate border of the Atlantic between Long Island Sound and the sea, including also a part of Staten Island. It is the northerly extension of the great coastal belt which, farther southward, becomes much wider and much more prominent as a physical unit. This belt is characterized by simple sedimentary strata of clay, sand, and gravel which lie almost flat, but really dip gently toward and beneath the sea. They are the simplest of the formations with which this work has been concerned, but are of prime importance, especially in connection with the water sources of Long Island. These great provinces differ strikingly in physical make-up, structure, and age. Each is complex, is structurally a group of physical units or formations distinguishable among themselves, and each presents its own peculiarities of behavior and geological significance.

Geological History.—The oldest of all the rocks are certain parts of the crystallines of the Highlands. These rocks are doubtless old sediments formed in the sea and by running water. They were a great series which was folded into mountain ranges in ancient times. Subsequently, they were invaded by immense floods of igneous material, which literally swallowed large parts of the solid foundations of that time and formed huge igneous masses now appearing as granites and other massive rocks. Rocks of intermediate composition were formed, doubtless by a mixture of the two original types—the sediments and the invading igneous material. This is well shown on the geologic profile, Plate II, from the Catskill Mountains to New York City along the aqueduct line.

These changes took place at great depth. The superficial formations of that time have been carried away by erosion. Only the roots of this ancient mountain range are now exposed, and they are the crystalline rocks of the Highlands. How much erosion took place no one can say, but after ages of erosion, so long indeed that the rivers of that time and the sea had cut away all the superficial material and had exposed the already cooled and crystallized igneous masses, the sediments that now form the other rocks began to accumulate. The land area lay along the Atlantic border and extended an undetermined distance into the present Atlantic Ocean. The sea lay to the west and in it were deposited the sediments carried down from this


CATSKILL MOUNTAINS
SCHOHARIE RESERVOIR
SHANDAKEN TUNNEL
ASHOKAN RESERVOIR
ESOPUS CUT-AND-COVER
PEAK TUNNEL
RONDOUT PRESSURE TUNNEL
BONTICOU
Elevation in Feet
2000
1000
0
−1000
Catskill Series of Sandstones and Shales
Ashokan Sandstones and Flags
Hamilton Sandstones and Shales
Marcellus Shales
Onondaga Limestone
Esopus Shales
Helderberg
Limestones
Shawangunk Grit
This portion of the profile is not drawn to scale
0 Miles
Binnewater Sandstone
High Falls Shale
10
CROTON LAKE
SARLES TUNNEL
REYNOLDS HILL TUNNEL
KENSICO RESERVOIR
EASTVIEW TUNNEL
ARDSLEY CUT-AND-COVER
1000
0
−1000
Schist
Diorite
Gneiss
Fordham Gneiss
Gneiss
Manhattan Schist
Inwood Limestone
70 Miles
80

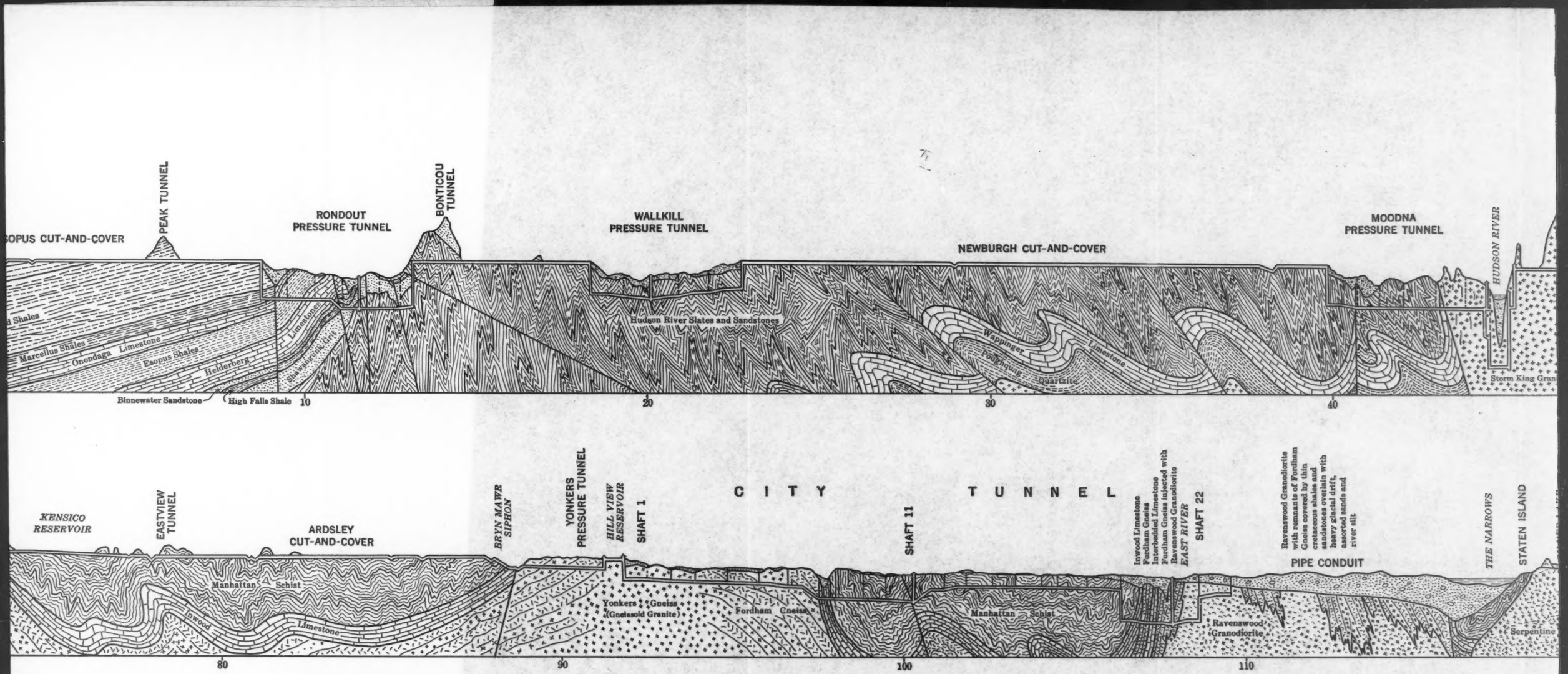
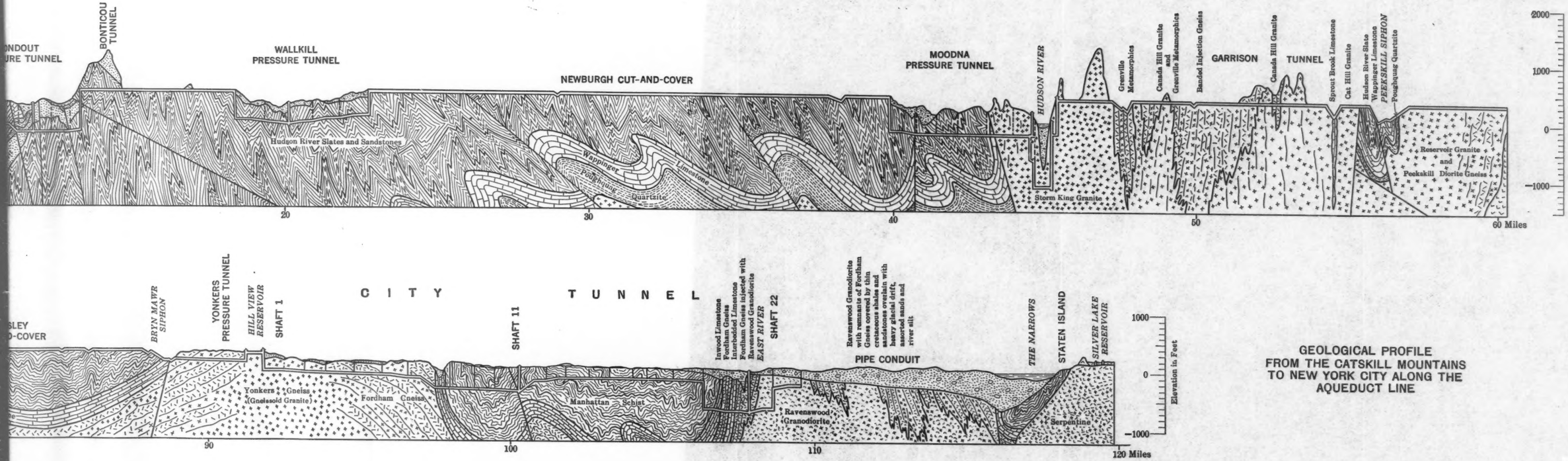

SOPUS CUT-AND-COVER
PEAK TUNNEL
RONDOUT PRESSURE TUNNEL
BONTICOU TUNNEL
WALLKILL PRESSURE TUNNEL
NEWBURGH CUT-AND-COVER
MOODNA PRESSURE TUNNEL
HUDSON RIVER
Shales
Marcellus Shales
Onondaga Limestone
Esopus Shales
Helderberg
Limestone
Shawangunk Grit
Hudson River Slates and Sandstones
Wappinger
Poughquag
Quartzite
Limestone
Storm King Gran
Binnewater Sandstone
High Falls Shale
10
20
30
40
KENSICO RESERVOIR
EASTVIEW TUNNEL
ARDSLEY CUT-AND-COVER
BRYN MAWR SIPHON
YONKERS PRESSURE TUNNEL
HILL VIEW RESERVOIR
SHAFT 1
CITY TUNNEL
SHAFT 11
Inwood Limestone
Fordham Gneiss
Interbedded Limestone
Fordham Gneiss injected with
Ravenswood Granodiorite
EAST RIVER
SHAFT 22
Ravenswood Granodiorite with remnants of Fordham Gneiss covered by thin cretaceous shales and sandstones overlain with heavy glacial drift, assorted sands and river silt
PIPE CONDUIT
THE NARROWS
STATEN ISLAND
Manhattan Schist
Inwood
Limestone
Yonkers Gneiss
(Gneissoid Granite)
Fordham Gneiss
Manhattan Schist
Ravenswood Granodiorite
Serpentine
80
90
100
110

PLATE II.
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GEOLOGICAL PROFILE
FROM THE CATSKILL MOUNTAINS
TO NEW YORK CITY ALONG THE
AQUEDUCT LINE

land area, first the sandstone, limestone, and shale of the present Hudson Valley, and after several thousand feet of these sediments had accumulated a new mountain folding and another period of erosion took place.

When deposition began again, it formed the great series of sediments that now lie above the Hudson River shale series. The Shawangunk conglomerate was the first, and on it were accumulated, one after another, the shales, limestones, and sandstones that now constitute the rocks of the Rondout Valley and the Catskill Mountains. Then came the Appalachian Mountain-making period, the last great folding movement to affect this region. Since that time, conditions have been completely reversed. The region to the west became a continental area and the re-adjusted drainage from this new continental area carried the sediments of subsequent time toward the east into the Atlantic. A long period of erosion followed, and, with the shifting of levels, the Atlantic gradually encroached from the east, covering a part of the old land area that originally furnished most of the earlier sediments.

In this long period of erosion and unchanged sea level, there was opportunity for the streams of the new Continent to wear the whole region down almost to a plain. The development in this "almost-plain" took place in the Cretaceous period, and, therefore, is referred to as the "Cretaceous peneplain." During part of this time and, subsequently, sediments have been accumulating on the Atlantic Coast. These are simple beds that are not much distorted. The Cretaceous peneplain still persists in a somewhat fragmentary form, because of much subsequent erosion. From a high elevation it can be plainly seen in distant landscape, as, for example, in the level sky-line of the Hudson Highlands, the Shawangunks, and parts of the Catskills, and more definitely in the remarkably uniform seaward slope of the crystalline rocks south of the Highlands passing under Long Island out beneath the sea. Its existence is shown by the boring profiles across Long Island to depths of 1 000 ft. and more below sea level, where the soft Cretaceous beds of the Coastal Plain lie on the old erosion floor of that time.

An elevation of the Continent of from 1 000 to 2 000 ft., during the Tertiary period, gave the streams a steeper slope toward the sea, with greatly increased velocity and cutting power, so that the old Cretaceous surface was deeply dissected by the streams cutting new valleys along the more easily eroded rocks. In the course of time, a new base level of erosion was reached in places where the rocks were comparatively soft, such as limestones and slates. This new peneplain, however, was never finished. The region was elevated again before the erosion was completed, and new trenches began to be cut by the streams. Traces of this Tertiary peneplain are readily seen, however, in the rather even stretches of ground, at 300 to 500 ft. above sea level, in the broad valleys of the Hudson and its tributaries. The importance of the questions previously discussed relates closely to the problems in the location of the Catskill Aqueduct, in the depths of old buried stream valleys and their effect on the location of tunnels. The Tertiary peneplain was a feature of great significance in determining the course and hydraulic grade of the aqueduct. The 15-mile section of cut-and-cover aqueduct from the Wallkill to the Hudson River is made possible at its present grade by the

existence of this old peneplain. The subsequent continental elevation that ended the development of the Tertiary peneplain and caused the renewal of trenching, finally started the glacial epoch, and an entirely new set of conditions were thus established, which led to changes of far-reaching importance. This elevation probably ultimately amounting to several thousand feet, at first simply increased the cutting power of the streams and resulted in renewed trenching of the valleys, with the development of rather narrow deep gorges, extending far below the general Tertiary base level. These gorges still remain beneath the glacial drift cover, some of them extending several hundred feet below sea level.

The Glacial Period.—The ice invasion that followed, covered the entire region, including the Catskills and the Highlands, the ice occupying even the highest summits and spreading southward to Long Island. There, the ice melted, forming the Terminal Moraine which extends the length of the Island and crosses the Narrows to Staten Island, thus marking the southern limit of the great continental ice sheet. At the height of its activity, as it moved southward, the planing action of the ice mass smoothed the tops of hills and ridges, ploughed deeper in some of the softer material of the valleys, and scraped hard against all exposed northerly slopes. At the same time, particularly as the ice finally thawed, the deeper canyons and valleys were buried with glacial drift, thus modifying greatly the surface contour of the country and tending by these processes of erosion and deposition, to smooth the more rugged earlier topography. One of the results of this action is the buried channels and drift-filled gorges. The present streams scarcely anywhere flow in their original channels, nor even on the old valley bottoms. On this account it was necessary repeatedly to determine, by boring, the location and the depth of these old pre-glacial channels.

The damming of valleys by glacial ice and drift created lakes and gave opportunity for the development of silt deposits in them. Subsequent partial or complete removal of such barriers and final drainage of these temporary reservoirs are factors of important bearing in this region. The Ashokan Reservoir occupies, in part, one of these glacial lake basins. The material left behind by the ice sheet is a mixture of mineral and rock matter derived from the removal, transportation, and deposition of former residuary soils and partly disintegrated rock gathered up as the glacier moved slowly over the surface of the country. Some of it is a mixture of pebbles, boulders, sand, and clay, which was not assorted by the action of water. This is the so-called "glacial till", also known as "hardpan", when consolidated. Other parts of the drift were worked over by the water from the thawing ice and thus were more or less assorted so that the materials became separated as deposits of sand, gravel, and silt or clay. These deposits constitute the so-called "modified drift" deposits, which, unlike the till, are generally loose and unconsolidated.

It would be a comparatively simple matter to classify these glacial materials and judge their behavior if they were as distinct, continuous, and uniform from top to bottom as the average rock stratum. However, they are much more likely to be variable in every respect and the different types of these

materials grade one into another in a most confusing manner. They exhibit important differences in their capacity for holding water. Thus, the character of the drift becomes an important factor in the choice of location for dams and dikes. In fact, they were controlling factors in the location of the Ashokan Dam.

Ancient Drainage.—In a study of the possible crossings of the Catskill Aqueduct under the Hudson and its tributary streams, the old courses of the pre-glacial channels and particularly their depths, were vital factors in the design and location of the pressure tunnels.

In this district there is only one main drainage channel, the Hudson River, to which all the other streams are tributary. These tributaries have attained their present courses by the general northeast-southwest strike of the rock formations, the different members of which are strongly contrasted in their resistance to erosion. It thus happens that whereas all the tributaries on the east side of the Hudson are direct, those on the west side are retrograde, that is, flow somewhat opposite to the direction of flow of the master stream and join it at an obtuse angle. The Wallkill River is a remarkable example of this. It rises within 40 miles of New York Bay, and flows northeasterly, following the formational direction for 75 miles, getting farther and farther away and finally reaching the Hudson nearly 90 miles from the sea. On account of the glacial drift cover, many of these stream valleys are heavily filled and the present stream courses diverge greatly from these former lines. Still, the old courses were of greatest importance in the problems of the aqueduct. Thus, it happens that a working knowledge of the origin and habit of the pre-glacial drainage has been of greater practical service than the simple statement of its features would lead one to expect.

The Catskill Mountain Water-Sheds

The selection of the Catskill Mountains as a source of the new water supply of New York City was made because this area was found to be the one which, by reason of its large, sparsely inhabited, and well-wooded mountain slopes, is certain to yield an adequate quantity of water of exceptional softness and purity. The site chosen for the main storage reservoir, 100 miles from New York City, in the southern edge of the Catskill Mountain area, is at such an elevation that the water from it can flow by gravity through the aqueduct and distribution systems, thus eliminating the cost of pumping. Because of the steep mountain slopes, the water runs rapidly down the rocky stream beds, encountering on its way almost no swampy ground or stagnant ponds. Even almost all the water that sinks into the soil and enters bedrock returns clear and pure to the regular drainage courses at the lower levels in the immediate district, because the beds lie nearly flat and have many interbedded tight shale members to obstruct the downward course of the water.

The gathering ground in the Catskills lies entirely in a region of sandstones and shales, without much admixture of limy fossil remains and no limestone beds. Such rocks are very resistant to solution and, for this reason, the water which flows over and percolates through them, dissolves so little mineral salts that a water unusually soft and pure is supplied.

The sedimentary beds consist of alternate layers of hard sandstone and soft shale, with wide variations in the quality and thickness of individual beds. In many places, the sandstone is of the quality known as bluestone, and has been extensively quarried. Generally, the strata dip to the west at a small angle, and there is no marked deformation. In a broader view, the region may be regarded as a deeply dissected plateau of which the rocks of different hardness, lying in beds dipping gently westward, give form to the back slope of the uplands, whereas the edge of the Catskills toward the Hudson Valley presents a pronounced eastward-facing scarp, rising abruptly several hundred feet above the valley floor. Erosion has been carried so far that the most elevated remnants have a mountainous aspect. Below these high peaks, the country is more open and uniform, being part of an old base level, or peneplain, judged to be the Cretaceous peneplain referred to previously. Deep canyons and high cliffs are common, with piles of broken rock at their bases. Glaciation has modified the original rugged features, especially the minor details to some extent, and glacial deposits have changed the drainage in many of the valleys, so that the present streams do not follow the courses of the old pre-glacial channels.

The dark sandstones, locally known as bluestone, with the associated shales, are simple sedimentary formations of great lateral extent and thickness. They were laid down as delta deposits, the sedimentary material varying from coarse sand and even small pebbles to the finest silt. The shifting of depositing currents and changes in the supply of materials resulted in a complex structural relation of the beds with frequent variations in the grade of the rock.

This variability in the quality of the rock within short distances is a striking feature in this formation, and was important in the selection of quarries for construction materials. The variation in the grade of the rock also has an important bearing on questions of value, as was illustrated in several large claims for damages where lands were condemned and taken by the City for the Ashokan site. The bluestone, shown in Fig. 3, is a rather fine-grained, dense, and impervious sandstone, very resistant to weathering. Blocks of the bluestone were used in the Cyclopean masonry of the Olive Bridge Dam, and the Gilboa Dam and Spillway will be faced with this material, which is as durable as the best granite. Bluestone is commonly used in New York City for curbs and sidewalk flags.

The Ashokan Reservoir

The main storage reservoir of the Catskill System has an available capacity of 128 000 000 000 gal. The principal structures which form this reservoir are the Olive Bridge Dam, built of masonry, across Esopus Creek, the earth dikes which close the gaps between the hills forming the sides of the reservoir, the dividing dike and weir, which separates the reservoir into two basins, and the waste weir.

The west basin of the Ashokan Reservoir in the Catskills, is shown in Fig. 4. The level top of the old plateau, deeply dissected by streams, is well shown; the stratified rocks dip to the west (left) at a low angle. The flow-line elevation of the reservoir is 600 ft. above tide and the maximum elevations



FIG. 3.—SANDSTONE USED ON CATSKILL WATER SUPPLY SYSTEM.



FIG. 4.—VIEW OF WEST BASIN OF ASHOKAN RESERVOIR.

of the mountains in this view are about 3 500 ft. Slide Mountain is 4 200 ft. in elevation.

Its geologic setting is as follows: The continental ice sheet, following the trend of the Appalachian Valley, moved in a southwesterly direction, as shown by glacial scratches. Local glaciers originating in the Catskills may have moved down through some of the main valleys such as the Esopus, but their effects are not very pronounced, and the general ice movement was transverse to this part of the valley. The character of the glacial deposits shows that the ice must have stood in the vicinity of Olive Bridge for a long time in the closing stages, forming an extensive dam, while the melting ice produced a glacial lake in the valley up stream from this dam, where silt and mud were deposited in the form of stratified sand and clay.

Heavy till deposits still form an effective dam at Olive Bridge, but the waters of the lake escaped along the western edge of the valley and cut through the drift, finally reaching bed-rock and cutting a new gorge in it. At the same time, the stream farther up the valley carried off great quantities of the soft and easily removed lake deposits originally laid down in the glacial lake, thereby further emphasizing the basin-like form of the reservoir site.

Geologic Conditions Affecting Dams.—The influence of geologic factors in the selection of dam sites is always considerable. A dam project is seldom too thoroughly studied or a site too carefully explored. Frequently, it is taken for granted that conditions are acceptable, which subsequent development prove to be dangerous. Expensive explorations, also, may be made, which intelligent judgment would show were unnecessary.

Studies of the dam sites of the Catskill project included investigation of the following subjects:

1.—Enough of the geology of the region to be certain that no important geologic influences were overlooked.

2.—Detailed topography over the site and adjacent areas.

3.—Bed-rock contour.

4.—Character of material overlying the rock floor, particularly in regard to porosity and behavior under construction disturbance.

5.—Character of bed-rock. Kind and quality of rock. Condition of the rock with respect to solution or decay or disintegration. Structural features, such as joints, bedding planes, faults, or crush zones. Porosity of the rock and nature of the water circulation, whether through the body of the rock or only along the bedding planes or joints.

6.—Exploration of underground conditions by borings or other means, with directions as to their location and order. Interpretation of boring data with tests of the ground through the borings themselves.

7.—Related questions. Local sources of material for construction, and comparison as to quality and cost, with other available materials. Need of treatment of foundation by grouting or otherwise. Probability of earth movements of such magnitude as to injure the structures. Possibility of claims for damages.

In the selection of a site for the Ashokan Reservoir several locations were investigated. The number of practicable sites was finally reduced to two, the "Olive Bridge" and the "Tongore" sites.

The Olive Bridge Site.—The nature of the drift in the trenches, shafts, and surface exposures of the vicinity indicated that the drift at the Olive Bridge site had been, at one stage in the glacial history of the locality, a natural dam and that water had been successfully held above it to an elevation of 530 ft., and perhaps more. The lowest materials in contact with the bed-rock are heavy stony till, laminated till, and stony laminated clays, all good impervious material making tight contact with the rock, and although a deep pre-glacial gorge lies underneath, at one side of the valley, it is filled with this tight drift, and there is no question whatever about its water-tightness.

Sands and laminated clays are extensively developed immediately up stream from the site. These looser materials grade into the heavy till material of the site proper and approach so close that occasional pervious streaks extend well into the westerly edge of the site. They do not at any point extend continuously through this ground, however, and, consequently, the present barrier, as it stands, is practically impervious. On account of the poorer quality of the drift up stream, the dam site was located as far down stream as engineering considerations permitted.

The Tongore Alternative Site.—At the Tongore site, the bed-rock floor in the buried gorge is at least 100 ft. deeper than at Olive Bridge. In the deeper parts, below the 400-ft. elevation, the deposits as indicated by the wash borings, are interpreted as a fairly continuous succession of stony drift, stratified sands and gravels, and laminated sands and clays belonging to two or three different early stages of accumulation. On this material, the heavy upper till was laid down. These overlying deposits are, as a whole, essentially impervious, but the lower 150 ft. will allow ready movement of water, especially laterally. This seems to be indicated also by the rather persistent occurrence of springs and soft wet places along the creek bank at about this level, both above and below the site.

Because of the higher bed-rock at the Olive Bridge site, the more uniform and impervious quality of drift deposits, the more massive cross-section of the barrier of drift on the north side of the site, and the glacial geology connected with the development of these features, that site was considered the preferable location for the dam on Esopus Creek. After the site was selected, the rock beneath the proposed dam was explored in detail by numerous core borings, and the holes were tested for the purpose of locating the position and permeability of seams in the rock. The bearing power and permanence of the blue-stones and shales beneath the heavy masonry structure and the possibility of creep of the softer shale members were also investigated.

The Schoharie Development

To complete the Catskill Water Supply System, the Schoharie water-shed is being developed by the construction of the Gilboa Dam which will form the Schoharie Reservoir. From this reservoir, the water is to be diverted

through the Shandaken Tunnel, now under construction, and will flow down the channel of Esopus Creek into the Ashokan Reservoir. Schoharie Creek lies on the north side of the divide from Esopus Creek, in the higher region of the Catskill Mountains. The Esopus flows out of the Catskills through the southerly gateway toward Kingston, N. Y., and the Hudson River, and the Schoharie flows north to the Mohawk River, near Amsterdam, N. Y. The water-shed, 314 sq. miles in area, is similar in character to that of the Esopus, chiefly steep mountain slopes covered with forest growth. The underlying rock consists of sandstones and shales through which the 18-mile Shandaken Tunnel is driven.

Several sites were investigated for the location of a dam on Schoharie Creek. The controlling geological features in the selection of such a site were the relation of a deeply buried inner gorge cut down into bed-rock by the interglacial stream and the quality and structure of the glacial drift deposits above the rock.

The Devasego Falls Alternative Site.—Of all the sites examined, that at Devasego Falls at first appeared to be the most promising from the standpoint of economy, on account of the favorable topography. At this location, a ridge of glacial drift extends into the main valley from the west and seems to have formed part of a dam across Schoharie Creek in late glacial time, causing a lake to form on the up-stream side to the south. The modern stream has trenched through this drift ridge, but at this point it is lodged far to the east side of the valley, where it soon cut into the rock of the valley side and is still cutting in this narrow post-glacial gorge with its inevitable water fall. This is known as Devasego Falls.

As the stream cut deeper and deeper through the drift obstruction, it drained the lake and has since carried out most of the loose fine sediment which formerly partly filled the basin. This easy erosion, together with the usual tendency to meander where the stream was free, has widened the valley above Devasego considerably, whereas the stream at the proposed dam site had become lodged in a rock cut at one side and the valley bottom was left still heavily obstructed with drift. It was proposed at first to take advantage of this natural barrier of glacial drift, which crosses and almost closes the valley, and to form a reservoir by building a short dam to close the gap made by the modern stream. As the depth of water in the reservoir thus formed would have been nearly 100 ft., the main question was to determine the degree of porosity of the material in the rather narrow drift ridge which would have to retain the water. The ridge is essentially a "kame deposit", formed at the edge of the ice during a melting stage. The material is, in part, transported and assorted and must have been laid down by water flowing from the mass of melting ice, and, in part, it is residual till probably frozen originally in blocks of ice, left to thaw in their present position. The whole is capped by impervious clay, a remnant of the later lake deposit. Formerly, the clay was undoubtedly much more extensive, constituting the topmost beds formed in the standing waters, but the stream has carried it nearly all away, except on top of this protected ridge.

It was anticipated that the mixed materials deposited by both water and ice would be in part porous and in part impervious. This condition was found on examination of the dry samples taken at frequent intervals in the borings. Furthermore, the sandy, porous material and the dense, impervious parts of the ridge were found to lie in no regular relation to each other. At first, it was hoped that these materials of different qualities might be so interlocked and so limited in lateral extent that no strata of porous material would extend continuously through the ridge. It was observed, however, during the drilling operations that the behavior of the ground-water was unusual. The water-table was very low and did not maintain even the general elevation of Schoharie Creek on the up-stream side of the ridge. There was also a tendency to lose water below the water-table as freely as above it. As the imperviousness of the ground was a vital factor, it was decided to investigate further the porosity of the material and the underground circulation by pumping tests. For this purpose, the casings were left in many of the holes. Water was pumped into one hole continuously for a period and the rise in the water was noted in the other holes. Prior to testing, all the casings were lifted from the rock floor to the first pervious layer, as indicated by the original records of each hole, and as the casing was raised the space was plugged with clay. This prevented the water from escaping down to and into the rock which was badly fissured. Water was then pumped into a centrally located hole, and the position of the water-table in the other holes was ascertained by soundings.

The geological sections previously developed by the borings had indicated certain pervious zones, and the results from the pumping tests confirmed the existence of these zones and, in addition, made it possible to trace them more or less successfully either to their limits or, at least, to establish their continuity. In general, a broad and extensive pervious zone, 1 000 ft. wide, 1 200 ft. long, and 30 ft. thick, the top of which is about Elevation 1 100, was found to extend in a northwesterly direction, reaching through the ridge to the under strata of the Bearkill. As a result of this investigation, and for the additional reason that investigations at the alternative site at Gilboa have shown decidedly better conditions there, the Devasego site was definitely abandoned.

The Gilboa Dam.—After investigating several favorable locations on Schoharie Creek, it was decided to locate the dam at Gilboa, N. Y., and create a storage reservoir in the valley of that stream. A line of borings across the valley showed that the bed-rock on the east side of the river is thinly covered with glacial drift and slopes gently to the west, and that the west side of the valley is filled with compact glacial till which fills the pre-glacial bed-rock gorge with impervious material. Although here, also, there is a much deeper old rock gorge beneath the drift of the west side of the valley, the natural obstruction now filling it, represented by the clay and till, is so effective a barrier that advantage can be taken of it in constructing the dam.

At Gilboa (Fig. 5), as at Olive Bridge on Esopus Creek, the present stream flows in a new rock channel in the side of the valley, made since the ice of the glacial period withdrew, and the old channel is buried under a

GEOLOGIC SECTION ON CENTER LINE OF PROPOSED DAM

Fig. 5.

great load of compact material. This is the ideal location for a dam in glaciated regions, but it is seldom that better conditions, even of this kind, are found than those presented at Gilboa. To meet the conditions, the dam at Gilboa will consist of a masonry section of Cyclopean concrete founded on rock in the present stream and the east end, and will join the till barrier of the west bank by means of a substantial earth dam with a core-wall deeply seated in the compact earth. As the form of the valley is a deep and narrow trench, the masonry section of the dam will be constructed as an over-fall spillway. The down-stream face will be made in large steps faced with natural stone. The face of the Ashokan Dam is formed of concrete blocks. The durable bluestone bed-rock of the gently sloping east bank will be stripped to form the over-flow channel, and the water will thus flow down the dip slope of the beds, parallel with the face of the dam, discharging into the gorge.

At the several quarries investigated, there are beds, 2 to 3 ft. thick, suitable for supplying blocks for the facing of the dam. In a general way, all of them are more quartzose than the average bluestone and some, therefore, are more nearly related to quartzites than to the simpler bluestones such as were used for the blocks in the Cyclopean masonry of the Olive Bridge Dam. There are no constituents that readily decay or develop stain or are likely to exhibit weakness. The mineral composition is as stable as the constituents of the best types of rocks.

The rock has a roughly granular structure and undoubtedly was a very granular accumulation when it was deposited. Microscopic examination, however, discloses that many of the original grains are fragments of rock rather than fragments of simple minerals, and, therefore, are complex in their own structural make-up and mineralogy, each fragment containing quartz, sericite, and sometimes chlorite. This fact makes the rock look more complex than the usual sandstone, but all the fragments are of durable, firm materials and no detrimental quality is introduced by the fact of their complex structure. It would be difficult to find better rock for exposed structures than the bluestones of the Catskills. Even a granite resists weathering no better, and some varieties of granite, because of their different structure, are not as durable.

Location of the Catskill Aqueduct

In the early investigations of a water supply for New York City, sources east of the Hudson River, as well as the ground-waters of Long Island, were considered. As these studies were extended, it became evident that the streams of the Catskill Mountains were the best available sources. A storage reservoir site of great promise was shown to exist at Ashokan, and rather early in the studies it became evident that the aqueduct must start from the Ashokan Basin. The problem was one of conveying a large quantity of water from the Esopus water-shed to New York City. It was also necessary to develop a considerable storage basin near the city, on the line of the aqueduct, and, for this, the Kensico Reservoir site was selected.

The region traversed by the aqueduct may be divided, for convenience, into the following districts:

(1) The elevated easterly margin of the Catskill Plateau, extending from the Ashokan Reservoir to the Rondout Valley;

(2) A part of the Great Appalachian Valley, extending from the Rondout-Esopus border of the Catskill Mountains to the Highlands of the Hudson;

(3) The mountainous Hudson Highlands;

(4) The Piedmont region of Westchester County, south of the Highlands, extending to Long Island Sound; and

(5) Part of the Coastal Plain, represented by Long Island.

Each of these types of country presented problems peculiar to its own structure and history, so that a great variety of factors is involved. The type of structure selected and economy required that as much of the aqueduct as possible should be constructed on the hydraulic grade, but where the country rises above the hydraulic grade, it was clear that such heights must be penetrated by tunnels at grade, whereas valleys or depressions could be crossed by pressure tunnels at considerable depth in sound rock.

Geologic Features Affecting Location.—The geologic factors entering into the location of the aqueduct take into account such large features as the structural basis and origin of the topography with its variety of relief forms, modifications of drainage, and changes in topography due to glaciation, the general structural trend of the formations, with their attendant folds, faults, joints, cleavage, and the general physical condition of the rocks of the region. All these geological factors contribute to an understanding of the present and former topography and the general rock condition and behavior of the ground. They affect, therefore, the general problem of location. For example, the Tertiary peneplain in its gradual descent from the inland country toward the sea practically fixes the hydraulic grade across the broad tracts bordering the Hudson River. In like manner, the trend of formational structure is in direct control of the courses of the valleys and ridges, and thus, also, governs the choice of possible courses the aqueduct might follow.

The first aqueduct route to be investigated began at the east end of Ashokan Reservoir near West Hurley, N. Y. Shortly after this line was surveyed, detailed studies showed that a change in the out-take of the reservoir from the location studied near West Hurley at the east end to a point near the Olive Bridge Dam had certain advantages in the operation of the aqueduct. In addition, the shifting of the line farther west made available certain topographic features favorable to its location. Investigations by borings in the Hudson River near New Hamburg, N. Y., early established the fact that the rock gorge there is very deep. Furthermore, on the New Hamburg route, a crossing of the Hudson Valley from the vicinity of Marlboro, N. Y., to the Highlands near Fishkill Village, would have required a pressure tunnel about 9 miles long, penetrating the limestone beneath the river. The uncertainties of behavior of the limestone under such conditions early led to suspicion of all lines along which these dubious formations were prominent. In addition, several faults would have to be crossed by such a tunnel, and it was believed that some of them might prove to be troublesome either in construction or maintenance. Consequently, attention was called to the possible advantages of crossing the stream between Storm King and

Anthony's Nose where the old crystalline rocks of the Highlands assured a much firmer medium for a tunnel and the avoidance of most of the uncertainties and known difficulties of the New Hamburg route.

Choice of the Storm King Line.—The work of exploration was finally concentrated at Storm King on the northern margin of the Highlands, where the river was narrow, the rock sound, and the conditions generally favorable for deep shafts. Preliminary to the adoption of this location, four lines were investigated, namely, the Storm King, the Little Stony Point, the West Point, and the Anthony's Nose lines.

The objection to the Anthony's Nose line was that a great part of the route through the Highlands would be in tunnel, with several deep shafts, in places far back in the hills difficult to reach, with a long pressure tunnel beneath the river itself. On the West Point line, although the river is only 2 100 ft. wide, the distance between hydraulic grade points is 1.6 miles. Furthermore, a wide crush zone and a belt of ancient limestones are believed to lie beneath the river, which was judged to be less favorable for tunneling than a more substantial rock. The Little Stony Point line had the disadvantage of requiring some cut-and-cover on the west side of the river, in glacial drift, along an exceedingly steep mountain side, where land slides of soft material might endanger the stability of the aqueduct. On the route finally chosen at Storm King, the river is 2 900 ft. wide, with excellent rock on both shores. Favorable and accessible shaft sites near the edge of the river were an important advantage in this location, and the "cut-and-cover" country on the east side of the river through Breakneck Valley made for additional economy. The determination of the depth to bed-rock under the river was a critical problem by any route, as that depth would control the depth of the pressure tunnel. It thus became practical to concentrate work at a definite point so that this investigation could be made. Boring operations for this purpose, in the gorge of the Hudson, were carried on for about two years. This location is described subsequently in more detail.

After the Hudson River crossing was once established, the route to the south of it became fairly well fixed, the first objective being Kensico Reservoir and the second one, Hill View. The controlling features were the topography of the region and the economy of the project. No sufficiently objectionable conditions were found to warrant great changes of course after it was once laid out. South of Hill View, however, within New York City proper, many alternatives were presented, which required detailed study and exploration. This part, which is known as the City Tunnel, is discussed subsequently in more detail.

Cut-and-Cover Aqueduct South of Ashokan Reservoir

From the main head-works of the Ashokan Reservoir at Brown's Station, N. Y., the aqueduct follows the post-glacial valley of the Beaverkill, along a drumlin, as a cut-and-cover structure as far as the gorge of Esopus Creek which cuts at right angles to the course of the aqueduct. On the north side of this gorge, a boring was made to determine the depth to the bluestone ledge which outcrops in the stream bed and across the valley of the creek to the south.

It was expected that this valley would be crossed by a pressure tunnel deep in the rock, but as the borings in the north bank showed that the bed-rock was deep and sloped downward to the north beneath the drift hills into the buried gorge of the pre-glacial Esopus, the plan for a tunnel was abandoned, as the down-take shaft would be in earth to a prohibitive depth. A steel pipe siphon, therefore, was adopted instead of a tunnel in rock, for the crossing of the Esopus trench.

An inspection of the topographic map, particularly at Elevation 500, shows that the south side of the Esopus Valley, for a distance of several miles south of Ashokan Dam, is much smoother and less broken by tributary gullies than the north side. This difference is due largely to the effect of glacial action in smoothing the south bank in the southwestward movement of the ice during the main ice advance, whereas the roughness of the north bank is due to the gullying action of the glacial waters discharging from the northeast into the Esopus gorge in the retreat or melting stage of the ice. Because of this greater smoothness of contour, the south side of the valley was chosen for the location of the aqueduct. The only considerable depression on this line for several miles is the gully of Tongore Creek near Davis Corners, N. Y., where a pronounced trench is cut in the glacial till by waters discharging from the drift-filled valley to the southwest. It was crossed, as in the case of Esopus Creek, by a short steel pipe siphon.

At the "Atwood Cliffs", still farther south, the aqueduct was carried along the face of the precipitous escarpment for 900 ft. with a deep cut in the rock. In the much jointed, horizontally bedded shales and bluestones of the Hamilton, which form the bed-rock in this section of the line, it was found that the explosives used shattered the rock so badly that the trench broke wider than had been expected, thus increasing the work involved, particularly where the surface material was affected considerably by weathering.

Peak Tunnel was driven through the bold hill at the point of leaving the Esopus Valley, thus making a short-cut course to the much smoother country along the edge of the Hamilton escarpment which forms the westerly margin of Rondout Valley. For the cut-and-cover portion south of Peak Tunnel, hard, well compacted, glacial till was encountered, which was successfully excavated to the narrow base and steep slopes prescribed, effecting a material saving over the wider trench in the horizontally bedded and weathered rock north of Atwood, N. Y. At the southeasterly end of this stretch of cut-and-cover construction, the Rondout Valley is reached, and a pressure tunnel in bed-rock was adopted for this crossing.

General Considerations Relating to Design of Pressure Tunnels

Of all the types of aqueduct, the pressure tunnel, perhaps, was given the most study, as there was no precedent for tunnels operating under such high heads. Pressure tunnels had been used to some extent, but none of them was subjected to an unbalanced head much greater than 100 ft., whereas on the Rondout Pressure Tunnel the unbalanced head is about 400 ft. and the ground-water head on the Hudson Pressure Tunnel is 1 100 ft. when the tunnel is empty. The pressure tunnels are designed with vertical waterway shafts and a tunnel deep in the rock to carry the water from one side of the

valley to the other. The strength and water-tightness of the rock is mainly depended on to retain the water without undue loss. The tunnel excavation, which is circular in form, is lined with concrete, to serve not only as a support to the rock and to provide a high coefficient of flow, but also to obstruct the escape of water from the tunnel. This last function of the concrete is aided by grout, which was forced under high pressure into all openings between the concrete and the rock and also into all water-bearing joints and porous openings of the rock itself, within reach of the operation. The concrete lining, therefore, supplemented by the grout filling of all adjacent spaces is depended on to make a reasonably water-tight conduit. The advantages of this type of construction may be enumerated as:

(1) Great permanence;
(2) Very low maintenance cost;
(3) Cheaper original cost than any other equally efficient conduit; and
(4) Convenience in operation.

The last two considerations may be illustrated by the 18 miles of pressure tunnel in New York City where sixteen lines of steel pipe of large diameter would have been required to carry and distribute a supply equivalent to the single pressure tunnel.

Permanence is a matter that has been most carefully considered. In the case of other pressure tunnels where relatively low pressures have been used, the rocks are stable and little disturbed by folding and faulting. In the country traversed by the Catskill Aqueduct, serious local movements have disturbed the strata, but the rocks are generally of such an insoluble and permanent character that there is little fear of their deterioration. This sound character of the rock is a decidedly favorable condition. It appears also that the lower New York region is now comparatively stable as compared with most of the Rocky Mountain or Pacific Coast regions. One must recognize that movement might occur here, or for that matter anywhere else; but few known regions have so little evidence of recent disturbance as this, although in an earlier period such movements were common and extensive. Some of the most important effects of these earlier disturbances may now be seen in the faults and crush zones encountered in many places. At such points, the rock is much weaker than usual and is also less stable during and after construction, and, furthermore, it is believed that if a new movement should affect the region, it would be likely to follow some of these old weaknesses. Therefore, the line with the fewest known faults is the most desirable, not only because of the wish to avoid bad ground, but also because of possible further shifting.

In order to prevent bursting of the tunnel subjected to high pressures, which would result in leakage and loss of water, it is essential that the rock be strong enough to meet the severe demand made on it. Therefore, a proper understanding of the condition of the rock became of prime importance. Zones of disturbance, or decay of the rock, and zones of weakness induced by deformation were carefully considered. It was deemed to be very important to place the tunnel at an adequate depth beneath the surface of sound rock in order to escape the influence of superficial weaknesses. There-

fore, in all cases, a cover of not less than 150 ft. of sound rock was provided, except for comparatively light heads.

The circular shape was adopted for pressure tunnels and permanent shafts, because of the external pressure to which they may be subject, in places, when the tunnel is under construction or out of service. The pressure from ground-water depends on the relative porosity of the rock and the concrete. Thus, with a tight lining and porous rock, the lining would receive nearly the full ground-water head; whereas, with a tight rock and an equally impervious lining, the lining would get little pressure. It was thought that porous or seamy rock would be made tight by grouting and that the rock itself would constitute the main reliance against inward or outward leakage. The lining of the tunnel was made of such thickness as to withstand the static ground-water head and the grouting pressure, and the grouting of all openings back of the concrete lining was depended on to cut off outward leakage. This provision is very important, as water can escape wherever it can enter a tunnel and in much greater quantities, because the unbalanced heads producing outward flow from the full aqueduct under pressure are much greater. Ground-water pressures of 200 to 475 lb. per sq. in. have been measured by means of gauges applied to grout pipes in the tunnels.

The Rondout Pressure Tunnel

The comparatively low Rondout Valley is crossed by a pressure tunnel 4½ miles long, which was driven from eight working shafts about 500 ft. below the general floor of the valley. The tunnel has an inside diameter of 14½ ft., and the concrete lining averages 22 in. in thickness. The excavation averaged about 10 cu. yd. per lin. ft.

Summary of the Rondout Problem.—The main problem presented in the study of the tunnel location related to the proper depth of the tunnel in the rock, which, in turn, was influenced by the location and depth of the pre-glacial drift-filled gorges, and the attitude of the several rock formations penetrated, especially the location, thickness, and inclination of the Shawangunk grit with reference to its intersection by the tunnel. General studies and explorations, made to determine the choice of tentative lines crossing the valley at a number of places between Hurley and High Falls, showed that the geological formations at the location chosen were less disturbed by folding and faulting than at any other part of the entire valley, and other considerations made the High Falls location the most desirable. A comparison of the cost of rock tunnels with the cost of pipe lines and other structures, showed the tunnels to be the cheapest, even with shafts more than 1 000 ft. deep and allowance for special construction in case unusually bad ground should be encountered.

Earlier exploratory borings in the valley at Hurley revealed the rock in the bottom of the same ancient valley at a depth of 70 ft. below sea level, and showed that it is overlaid by a layer of sand about 220 ft. in thickness. This valley, following the Hamilton escarpment along the easterly margin of the Catskills, is bottomed by the Onondaga limestone. The line was finally moved several miles farther south, beyond the reach of the Esopus proper,

where a much heavier drift cover badly obscures the topography of the rock floor. From a study of the outcrops, applying the recognized principles of erosion effects on dipping strata, it was expected that the borings on the crossing at High Falls would discover a gorge in a corresponding position along the westerly margin of the valley at the border of the Hamilton flag formation. This was indeed the case. In addition, another gorge was disclosed by the borings near the middle of the valley, deeply buried, and so masked by deposits of glacial drift that its existence is not indicated in the present relief.

The first general geological cross-section of the valley, however, based on surface indications, was a fairly close approximation to the structure finally developed in much more detail by the borings. Two buried pre-glacial stream gorges, and the fact that the rock in places was broken by faulting and was porous, controlled the depth of the tunnel. Near the east end of the tunnel, also, a fold involving the very hard Shawangunk grit made it desirable to place the tunnel so deep that it would pass beneath the grit and lie entirely in the more easily excavated Hudson River shale below.

The geologic complexity of this valley is due chiefly to the number and variety of rock formations, including many beds of limestone, shale, and grit, and to their folded and faulted condition. On these strata of different resistances to erosion, a topography consistent with the detail of the structure has been developed. Where the more indestructible beds, such as the Shawangunk grit occur, there are ridges, and where the less resistant beds form the floor, erosion has cut deeper and there are valleys. The relief features of the rock floor, chiefly erosion channels, represent the work of streams due to the uplift preceding the Ice Age. These minor features, however, are much obscured by the drift. It is these later channels, especially their great depth, that became critical factors in determining the depth of the tunnel. The following formations are penetrated by the Rondout Tunnel, the Hamilton flags being at the top on the west side of the valley, and the Hudson shales, the lowest of the series, being on the east side of the valley:

	Thickness, in feet.
Hamilton flags and Marcellus shales	700
Onondaga limestone	200
Esopus gritty shales	800
Oriskany sandstone (not separately developed)	...
Port Ewen shaly limestone, including the Oriskany transition	200
Becraft crystalline limestone	75
New Scotland shaly limestone	100
Coeymans limestone	75
Manlius limestone, including the Cobleskill, and the Rosendale and Rondout cement beds	100
Binnewater sandstone	50
High Falls shale, including thin limestone layers	75
Shawangunk conglomerate	300
Hudson River slates, thickness unknown, probably more than	2 000
Approximately	4 775

Factors Affecting the Tunnel Grades.—The tunnel profile was fixed by the following requirements:

(1) To pass beneath the buried Kripplebush Gorge, at the west side of the valley, with a sufficient rock cover;
(2) To pass safely under the buried Rondout Gorge, near Station 500, in the center of the valley;
(3) To avoid, as far as possible, the Shawangunk grit, in the Coxingkill syncline, at the east side of the valley;
(4) To pass through the porous Binnewater sandstone in as short a distance as possible;
(5) To drive up-hill at a high angle through the heaviest water-bearing members for the purpose of draining the water away from the heading;
(6) To avoid driving up-hill on grades of more than 2% where lower grades are possible; and
(7) To have the entire tunnel drain toward the sump at the drainage shaft near Rondout Creek.

These requirements were met by placing the northerly half of the tunnel at Elevation — 100 in the vicinity of the most porous strata. There, a 15% incline through the wet ground connects with the southerly half which was put at Elevation — 250 so as to keep the tunnel below the Shawangunk grit and in the Hudson River shales.

Question of Solubility of Limestones.—The question of the possible solubility of the limestones was considered with great care. Certain caverns, due to the solution of these limestones, can be seen at the surface, and others were indicated by the borings, and it seemed not improbable that many solution channels may have been formed in earlier times to considerable depth in both the Onondaga and the Helderberg limestones. Fig. 6 is a view in the Rondout Siphon Tunnel, which shows the solution effects on bedding planes of Helderberg limestone, causing wild breakage at "mud seams", requiring 7 ft. of concrete in the roof.

If these limestones, which have proven to be soluble when exposed to ordinary meteoric water circulation, should yield in the same way to the tunnel waters, not only would the concrete lining be subject to disintegration, but the supporting rock immediately outside of them might be, in part, removed and the escape channels enlarged, seriously increasing the loss of water from the tunnel. These considerations led to the tunnel being placed below the zone most affected by surface-water circulation, avoiding as far as possible the most susceptible strata. It led also to the rejection of limestone aggregate in the concrete lining.

Factors Affecting Construction.—Under the subject of design, the general condition of the rocks of this region and their structural relations have been discussed. The hard or soft character, porous or fractured condition, and resultant permeability, the chemical changes to which they have been subjected, all have a bearing on the methods of tunnel driving and the design of tunnel adopted. Although the rocks are mostly hard, durable, and sound, the earth movements to which they have been subjected have resulted in the

formation of folds, joints, occasional faults, and crush zones. Such conditions may result unfavorably by causing an unsafe roof in places, requiring scaling or the use of temporary wooden timber or even permanent steel roof support. In other cases, small folds or intersecting fracture planes produce blocky and caving ground. In the soluble limestones, the circulation of underground water tends to dissolve parts of the beds and to produce openings by the enlargement of joint planes, with the result that the rock is weakened and, in places, cavernous. Conditions such as these required enlargement of the excavation in unsound rock, reduced the speed of excavation and the placing of concrete, and, in a few places, introduced an added cost for steel or timber roof support.

Since the lining has been placed, an opportunity has been given to observe the effect of ground-water on the concrete. The concrete shows no tendency to deteriorate and tests of the aqueduct in service have shown the pressure tunnels to be tight against outward leakage.

The Rondout Pressure Tunnel affords a good illustration of the influence of underground conditions on the progress of tunnel work. Fig. 8 is a progress diagram of the Rondout Tunnel and illustrates the influence of adverse geological conditions on the progress of the work. Where the rock was fairly uniform in quality, as in the shales and limestones, rapid and consistent progress was made in excavating, the average progress in these rocks being 350 ft. per month. Because of the hardness of the rock, the progress in driving through the Shawangunk grit was much slower, averaging 160 ft. per month. The longest delay in the construction of this tunnel was at Shaft 4 and in the tunnel between Shafts 3 and 4. The difficulties there were due to the large inflow of water in the Binnewater sandstone and High Falls shale. Shaft 4 was flooded several times and the sinking of it was delayed about a year on account of the troubles caused by water. An account of the sinking of this shaft has been given in a paper* by John P. Hogan, M. Am. Soc. C. E.

The large mud seams and broken cavernous ground encountered in excavating the tunnel south of Shaft 3 necessitated special methods in driving and lining. Special provisions were made there for reinforcing the tunnel with a steel shell. These extra precautions and the provisions made for curing the natural weaknesses caused much additional delay. Hydrogen sulphide gas in the Binnewater sandstone and High Falls shale made it difficult for men to work, as their eyes were affected by the gas, and to remedy this trouble, ventilation by blowers was provided. The acid water and gas also attacked the switchboard equipment of the electric pumps, making special protection necessary.

Fig. 7 is a view in the Rondout Pressure Tunnel, which shows the wet ground encountered in the High Falls shale and the Binnewater sandstone where these rocks were penetrated by the tunnel at the overthrust fault near Shaft 4.

Although the water-bearing beds on the incline gave so much trouble, most of the water entered in a stretch only 175 ft. long, half way up the incline

* "Sinking a Wet Shaft", *Transactions*, Am. Soc. C. E., Vol. LXXIII (1911), p. 398.



FIG. 6.—VIEW IN RONDOUT SIPHON TUNNEL, SHOWING SOLUTION EFFECTS ON BEDDING PLANES OF HELDERBERG LIMESTONE.

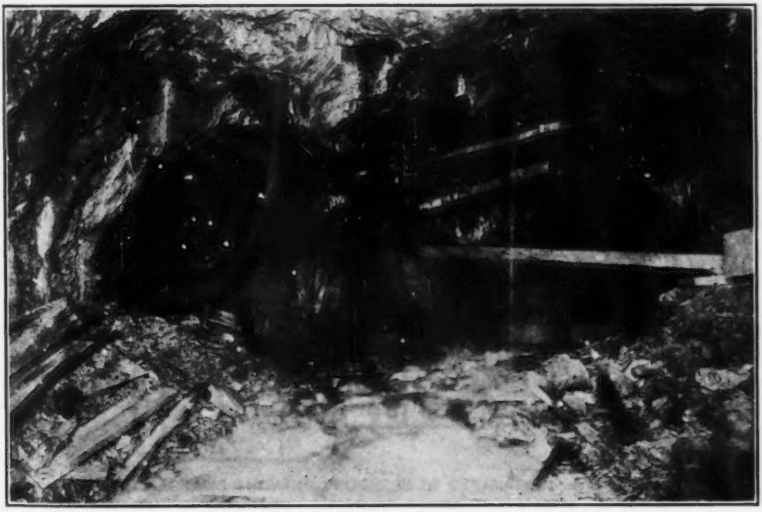
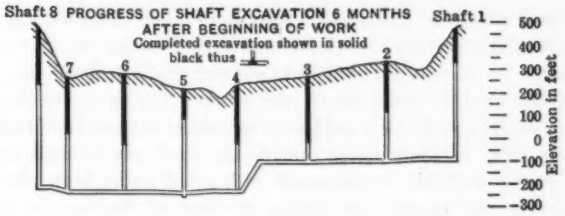
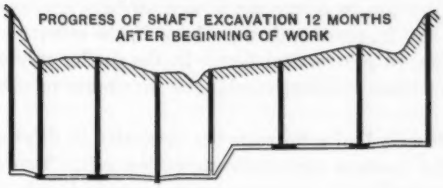
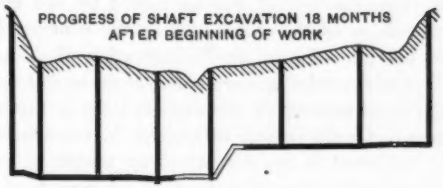
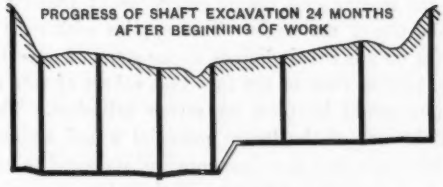


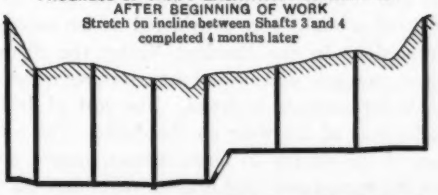
FIG. 7.—VIEW IN RONDOUT PRESSURE TUNNEL, SHOWING WET GROUND IN HIGH FALLS SHALE AND BINNEWATER SANDSTONE.











DIAGRAMS SHOWING PROGRESS OF EXCAVATION RONDOUT PRESSURE TUNNEL

The slow progress in sinking Shaft 4, and in excavating the Tunnel between Shafts 3 and 4, was due to (1) The excessive quantity of water encountered, (2) the effects of H_2S Gas and (3) the Crushed and Broken Ground Penetrated in the Tunnel.

FIG. 8.

in the High Falls shale, where these beds were much distorted along a strong thrust fault which seemed to furnish the way of escape for all the overlying porous beds. To prevent outward leakage from the tunnel which is subject at this point to a head of about 700 ft., the concrete lining was made 3 ft. thick. This lining was placed inside a steel shell to protect the concrete during placing and setting. Dry packing placed outside the steel was grouted and the surrounding porous rock was impregnated with grout under high pressure. The water entering the tunnel just before closing all leakage in this stretch amounted to 1 000 gal. per min. Ground-water was drawn down to Elevation — 55, and after all pumping was stopped, it took 40 days to restore it to Elevation + 65, a rise of 120 ft., representing a volume in the tributary underground reservoir of 64 000 cu. ft. per vertical foot. In the earlier stages of construction, the inflow was much heavier, reaching a maximum of about 1 900 gal. per min.

Although 250 000 cu. yd. of solid rock was excavated in driving this tunnel, none was suitable for concrete aggregate except the grit. Two quarries were opened in the Shawangunk grit outcrops situated east of Rondout Creek, making long hauls necessary in transporting the crushed stone to the several shafts. The rock excavated from the tunnel was examined by the geologists, who investigated the permanence of the stone used in the concrete lining of the deep tunnels and the possible solution or disintegration of the material composing the concrete. Their conclusions are summed up as follows:

The pressure tunnels are necessarily the weakest links in the whole Catskill System. They present not only special difficulties in construction, but they will be continuously subjected to greater strain by reason of pressures, and greater attack from the solvent action of water than other parts of the aqueduct line. If the work is to be permanent or even reasonably durable, every consideration affecting the matter as a public enterprise of importance is on the side of caution in the use of materials. This applies with most force to the pressure tunnels. Not only is it injudicious to use stone of uncertain or doubtful resisting quality, but, in view of the fact that others of safe character are available, such action would be open to severe criticism. The additional expense involved in the use of the better material would amount to such a small proportion of the whole cost as to lose much of its seeming importance.

The weak shales were condemned because of their low strength and marked tendency to dry out and crumble on exposure to the air. The limestones, although strong and hard enough, were considered as too susceptible to solution. Of all the formations in the Rondout Valley the Shawangunk grit, because of its hardness, strength, and insolubility, was regarded as the best of the local rock available for concrete material. The cost of drilling this very hard rock was high, because of the wear on the drills. The rock is strongly bound, and came out of the quarry in large masses, jagged and difficult to handle. The wear on the manganese steel crusher parts exposed to the cutting action of the hard quartzites was great, and renewals were frequent.

Failure of the Rock Under the First Hydrostatic Test.—After the tunnel had been completed, and the construction shafts sealed with concrete plugs,

the tunnel and end shafts were filled with water and subjected to the working pressure. Under this test the concrete lining was ruptured for several hundred feet between Shafts 3 and 4, making repairs necessary.

The tunnel lining had been cracked where the cavernous ground in the Helderberg limestones had been encountered, and also on the incline in the Binnewater sandstone. The limestones in particular had been weakened by solution along joints and bedding planes. These caverns and mud seams are not isolated, but are doubtless connected and irregular, following the pattern of the original joints. Evidently, the pressure in the tunnel had overcome the resistance of the surrounding rock, which was not everywhere solid and continuous, but in effect had become, in some places, a series of blocks separated by soft and yielding clay. It was necessary, therefore, to repair the damage and make the tunnel water-tight. Steel shells had been used originally at certain points of well recognized rock weakness, and at none of these places was there any movement or damage. Accordingly, it was decided to repair about 700 ft. of the cracked tunnel in a similar manner. This was done in three stretches by reinforcing the lining with a steel shell about 1 in. thick, made up of rings composed of 15-in. channels with their flanges riveted together. The cracks in the rock caused by the movement of the ground were then grouted under high pressure, to prevent the escape of water. This repair was successful and no further difficulty has developed. When the tunnel was designed, the possibility of such failures was recognized, but it was thought better to discover points of weakness by test, and to strengthen those parts that required it, than to reinforce all parts of the tunnel about which there might be some question.

The Wallkill Valley

From Shaft 8, the terminus of the Rondout Pressure Tunnel at the western slope of the Shawangunk Mountain, the aqueduct pierces the mountain about Elevation 450 by a tunnel which is driven in the easily excavated Hudson River shale beneath the cap of hard grit covering the north end of the mountain ridge. This tunnel emerges below Bonticou Crag on the east slope at a favorable point for a portal. A tunnel through this mountain at any other location would have been much longer, or would have encountered the resistant grit bed, in either case greatly increasing the difficulty of driving and the cost.

Freer Cut.—On the eastern slope of the Shawangunk Mountain (Fig. 9), where the rock ledges are sparsely covered by glacial drift, the aqueduct follows the contour southward at grade for a distance of 4 miles. Here, the excavation encountered the shales of the Hudson River series which were excavated rapidly and cheaply by blasting and steam shovels, except in the deep Freer Cut, where difficulty was encountered through a stretch of 2 000 ft. The strike of the rock formation here is nearly parallel with the center line of the aqueduct and the dip is about 40° in the direction of the general eastward slope of the mountain. (Fig. 10 is a view of the cut-and-cover aqueduct in Freer Cut, in the Hudson River shale, which shows wide breakage due to the dip of bedding planes at the west (right) side (1:1 slope). Note the steep slope (6:1) of the cut on the opposite side).

Trouble was had from the beginning on the west side of the Freer Cut by the sliding of the rock on its bedding planes. As the cut increased in depth, the shale slid down in great masses into the cut, burying the track and steam shovel. The right or east side was self-sustaining, because the dip of the beds on that side was away from the excavation. The greatest single slide amounted to about 2 000 cu. yd. of rock. Because of the great widening of the cut on the west side by these slides, it was necessary to re-locate the aqueduct structure after the excavation had been in progress for some months, moving the center line to the east so as to keep the excavation within the property lines of the city. The contractor's railroad for the stone supply from the quarry at Bontecou was also affected, and was rebuilt for a distance of several thousand feet. The conditions at Freer Cut increased the excavation from an estimate of 85 000 cu. yd. to 155 000 cu. yd. actual, besides requiring a substantial modification in the design of the concrete structure, in order to avoid possible damage.

Geology of the Wallkill Valley.—The Wallkill River is a striking example of a retrograde stream. It flows northeasterly from its source in New Jersey a total distance of about 75 miles in a direct line, to the southward-flowing Hudson which it joins at an obtuse angle. This fact alone is sufficient to indicate that its present channel is composite, and that a part of the drainage has been reversed in comparatively recent geologic time. A reversal of the drainage means that the modern river is at some point flowing across a former divide or water-parting; and the approximate location of this divide, if it is not too far from the normal course of the aqueduct, is of practical importance, as it would furnish a crossing with the highest possible rock floor and the least gorge development. The determination of the latitude of the buried divide and the location and depth of the pre-glacial gorge were given special study.

North of the New Jersey boundary, the geological structure of the Wallkill Valley is as simple as that of the Rondout Valley is complex, because from this section of the Great Valley, erosion has removed the entire series of Silurian and Devonian strata down to the Hudson River slate and sandstone. On the northwest, the Hudson River slate rises in the steep slope of Shawangunk Mountain capped by the heavy protecting bed of Shawangunk grit. Much of the floor of the Wallkill Valley is referable to the Tertiary base level; but north of New Paltz, N. Y., the immediate or inner valley of the Wallkill, 1½ or 2 miles wide, close to the base of Shawangunk Mountain is sunk below this level. To this inner valley, the Tertiary peneplain holds the relation of a broad uneven terrace stretching between it and Marlboro Mountain to the east. The Hudson River beds are, throughout, strongly folded and characterized by a marked cleavage structure dipping to the east. Throughout the area of the Hudson River formation, there is less agreement between the topography and the geological structure than with the other strata, but even in this series, wherever marked differences of hardness exist, this fact is reflected in the topography.

The absence of much boulder clay in this part of the valley greatly facilitated the making of test borings. South of Libertyville, N. Y., however,

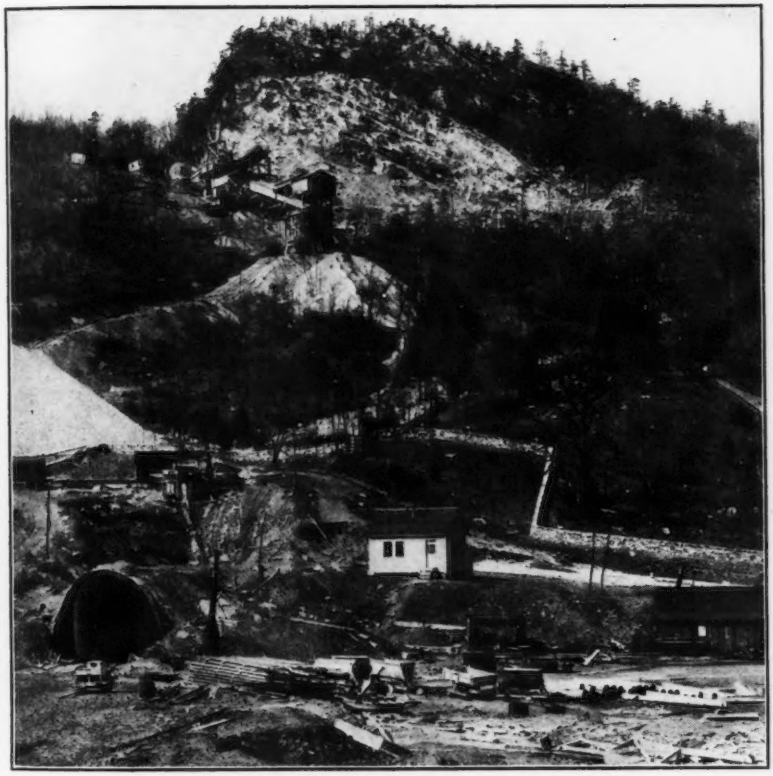


FIG. 9.—GRADE PRESSURE TUNNEL EMERGING BELOW BONTECOU CRAG, ON EASTERN SLOPE OF SHAWANGUNK MOUNTAIN.



FIG. 10.—VIEW OF CUT-AND-COVER AQUEDUCT IN FREER CUT.

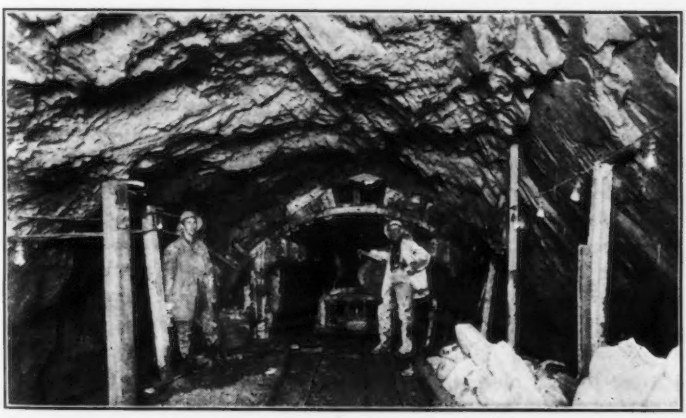


FIG. 11.—VIEW IN WALLKILL PRESSURE TUNNEL, SHOWING STRUCTURE OF HUDSON RIVER SHALE. CONCRETE ARCH FORMS IN BACKGROUND.

the drift is largely boulder clay for about 8 miles, to the vicinity of Walden, N. Y. The favorable character of the Hudson River shales for aqueduct tunnel construction was always recognized, because of the sound unweathered condition of these sandy mudstones. Although folded and faulted considerably, they are largely self-healing and tight, even along zones of strong movement. Almost the only purpose of the borings was to determine the position of the bed-rock floor and the depth and position of the buried pre-glacial channel of the Wallkill. The first and most northerly line of borings was made at Springtown, N. Y., where the old buried bed-rock gorge was found near the Wallkill Valley Railroad Bridge and was shown to be at a depth of 270 ft., extending 79 ft. below sea level. Southward, the elevation of bed-rock in the old stream bed rises rapidly until, at Libertyville, the bed-rock floor was at Elevation 67, or a rise of 147 ft. in 5 miles, an average of about 30 ft. per mile.

Design and Construction of Wallkill Pressure Tunnel.—The design of this tunnel was simplified by the fact that the rock to be penetrated was entirely Hudson River shale of almost uniform character. It was not necessary to consider, as in the case of the Rondout Tunnel, the rates of excavation progress to be expected, and the difficulties to be encountered in various broken and porous formations. The principal consideration in establishing the depth below the bed-rock surface was to provide adequate rock cover along the entire length of the tunnel.

The probable depth to which the rock might be decomposed and the general strength of the rock as it lies in the ground were also regarded as of prime importance. This kind of rock is not deeply affected by weathering, and although the ground-water level was doubtless much lower in pre-glacial times, there is no evidence that the rock was materially weakened except superficially. The ice sheet removed nearly all the loose and decomposed rock of that time, leaving the surface generally fresh and polished. This shale formation may be regarded as generally rigid enough to sustain itself and yet sufficiently yielding to make many open seams or joints improbable at the depth of the pressure tunnel. The great body of gravel and sand filling the main part of the buried gorge is indicative of a large water storage and suggests caution. Even so, a cover of 150 ft. of slate would seem to afford ample protection to the tunnel, because at this deepest point the rock is likely to be exceptionally fresh and sound.

The tunnel was accordingly designed with a pump shaft in the rock, with a minimum depth in earth, just east of the buried gorge, for convenience in discharging the drainage into the Wallkill River when emptying the aqueduct. The tunnel grade beneath the lowest point of the bed-rock of the pre-glacial gorge was covered by 150 ft. of bed-rock. The drainage shaft for the Wallkill Tunnel was placed near the Wallkill River and the spacing of the five other shafts was balanced in accord with this location of the drainage shaft. Although six shafts were sunk for this 4½-mile tunnel, eight of them were found to be necessary for the more complex conditions at the Rondout Tunnel, which was about the same length.

The general uniform quality and attitude of the thin beds of the Hudson River shale, together with the absence of open joints or porous beds carrying

heavy flows of water, made for comparatively rapid progress and low expense in the construction of the Wallkill Tunnel. The contract price per linear foot in the Wallkill Pressure Tunnel, including shafts, was $146.31, compared with $248.46 per lin. ft. in the Rondout Pressure Tunnel. This difference in price was due mainly to the difference in the geologic structure in the two valleys. Fig. 11 is a view in the Wallkill Pressure Tunnel, showing the prevailing structure of the Hudson River shale and its habit of breakage in the tunnel excavation. Concrete arch forms may be seen in the background.

The only hard rock encountered was for a distance of several hundred feet near the south end of the tunnel. It was very hard quartzitic sandstone, and it required from 20 to 24 hours to drill the heading as against 4 to 5 hours in the shale. The water encountered during the excavation of this tunnel amounted to 500 gal. per min. entering through joints, with no great flows at any one place. It was noticed that the inflow remained about constant for the entire siphon during the working period. This was supposed to be due to the great depth of the tunnel below ground-water level, so that at no point was the water-table depressed to the tunnel level, and also because there was slow circulation and no abnormal conditions.

As the Hudson River shale from the tunnel was not considered a satisfactory aggregate for concrete, the specifications excluded its use, and the contractor opened a quarry in the Shawangunk grit at Bonticou Crag, at the north end of the line, the nearest source of concrete stone. This quarry proved to be an expensive source as it was situated high on the steep slopes, making access difficult; there was no level bottom for loading cars; the steep down-hill dip of the grit strata made it difficult to operate drills; and the hard and abrasive quality of the quartzite caused rapid wear in drills and crushers. The sand for the concrete was secured from the Rosendale sand plain near the Wallkill Valley Railroad track, a haul of 9 miles. Sand deposits near the line of the tunnel were explored, and although the material was generally coarser than the fine water-washed Rosendale deposit, it contained enough fine material to make it necessary to remove the silt by washing.

Hydrostatic Test.—When this tunnel was first filled with water to the hydraulic grade, the outward leakage was about 1 100 gal. per min. This leakage rapidly diminished and, after several years, it became as low as 30 gal. per min. for the 4½ miles of tunnel, the reduction being due probably to the closing of the fine seams in the slate. When the first test was made, the water came to the surface at several places, particularly from drill holes which penetrated to the tunnel level. These holes formed active springs between Shafts 1 and 2, and softened the drift for considerable areas, but they finally ceased to flow.

Cut-and-Cover Aqueduct in the Wallkill Valley

The Tertiary peneplain on the easterly side of the Wallkill Valley is an open gently rolling country, the average elevation of which adapted it for the construction of the aqueduct in open cut for a distance of 22 miles, extending

from the uptake shaft of Wallkill Pressure Tunnel to the northwest margin of Moodna Valley which is also crossed by a deep pressure tunnel. Through this intervening upland, which forms a base-leveled divide, the deposits left by the glaciers presented favorable and easily excavated materials for the aqueduct trench. In these 22 miles, there were only two deep and heavy cuts, although the line is remarkably direct. Such a condition is due to the perfection of the base leveling or planation in Tertiary time, that is, the wearing down of the country to a plane, due to the long continued and unchanging level with reference to the sea.

Throughout the stretch, the prevailing material overlying the Hudson River shale is compact, glacial till, frequently requiring the use of explosives, except for 3 miles at the northerly end, between Ireland Corners and New Hurley, N. Y., where the east side of the valley is covered with kame deposits, evidently laid down while the center of the valley was occupied by a mass of ice. For 3 miles the aqueduct trench was excavated in this washed gravelly drift, as far south as New Hurley, where the line swings to the eastward, leaving the soft deposits and continuing south for many miles in hard glacial till. This difference in quality of materials affected the type of construction, the compact drift proving the more economical in cost of construction. For example, at the north end in the loose, water-deposited ground, an aqueduct with a wide base was required, and the payment lines for excavation were fixed on a slope of 1:1; whereas, in the solid ice-compacted till to the south, the structure was built with a narrower base and the trench was excavated to side slopes of 6:1. The cost per foot of the completed aqueduct in the modified or loose drift was $58.73, whereas in the adjacent compact glacial till, the cost was $48.06 per ft.

The Moodna Tunnel

Before it was decided to cross the Hudson River somewhere within the Highlands, there had been no Moodna problem. A crossing in the open valley region such as that originally proposed farther north at New Hamburg, would have avoided the whole Moodna section. As soon, however, as the crossing of the Hudson became fixed so much farther south, the line of the aqueduct was thereby established, approaching the Highlands on the west side of the river, and the crossing of the Moodna Valley assumed considerable importance. The Moodna is a comparatively small stream entering the Hudson just north of the Highlands, with its lower course lying across the most direct aqueduct line. The nature of this tributary, together with the character of the material through which it flows, suggests considerable drift filling.

By the time this problem came to be considered, it was known that the Hudson itself was very deep, several hundred feet at least. It seemed reasonable, therefore, to expect that a tributary such as this would have cut down to a somewhat similar grade. It was also appreciated from the studies of the Wallkill Valley, that the pre-glacial Moodna had probably been a larger stream draining some of the present upper Wallkill territory. In that case, the ancient Moodna must have been a more important stream than

the present one and probably had a somewhat different course. All this argued for a valley or gorge of much greater prominence than the present surface channel, but what its forms and profile was, no one could tell. The alternative Hudson River crossings in the Highlands involved corresponding changes in the location of the Moodna Tunnel, the lines farther south allowing the Moodna crossing to be located farther up stream. The Storm King crossing, however, which was finally adopted, gave preference to a line across the Moodna at its farthest down-stream position and permitted comparatively little chance for shifting.

It was planned at first to reach the Highlands margin with a pressure tunnel and to connect with the Hudson River Tunnel by a cut-and-cover aqueduct at grade along the northerly face of Storm King Ridge. It was finally decided, however, to construct the Moodna Pressure Tunnel on a more direct line and to continue at a suitable depth the entire distance to the Hudson River. This plan made it desirable to locate the tunnel as far to the north, down stream, on the Moodna, as natural conditions would permit, and explorations were conducted to determine the facts in detail. The early borings were rather widely spaced, but they soon disclosed that the deeper valley floor was abnormally flat for a distance of at least ½ mile and was covered with 300 ft. of very bouldery drift. Such a profile raised much speculation, the opinion prevailing that considerably greater irregularity probably existed than the borings had yet revealed.

It was known at this time that the gorge of the Hudson was from 500 to 600 ft. deep, and, as the floor of the Moodna Gorge, only 2 miles distant, was shown by the borings to be only 50 ft. lower than the present water level of the Hudson, the argument was felt to be strong in favor of a deeper inner notch somewhere in the Moodna Valley floor. The borings showed that there could be no notch in the floor wider than 200 ft., and it was decided that a very deep canyon need not be considered. It is surprising to find so flat a valley bottom at this elevation, in view of the fact that the Hudson Gorge is at least 700 ft. deeper only a short distance away. The stream ought to have cut a deeper notch within this bottom. It is probable that at least two causes unite to develop this abnormally flat form. Glacial scouring doubtless aided, and, probably, also the ancient course lay more nearly parallel to the line of borings than that of the present Moodna. On the whole, however, the Moodna profile appears to be the most abnormal of any on the aqueduct line.

It was necessary to determine the character of the rock of the valley floor, in addition to the depth of the gorge, but this of itself did not require a very extensive series of borings. It was easily found that the chief formation was the Hudson River shales and sandstones and that although beneath them and above the basal gneisses, there must be 1 000 ft. of limestones and 500 to 600 ft. of quartzite, only very short stretches of tunnel would penetrate either formation. No quartzite was encountered in the tunnel excavation and only about 200 ft. of limestone. At places where these formations would normally be expected, they have been cut out on the

tunnel line by faulting. Only one stretch of ground exhibited a change in formations or any unusual structural relation. It was discovered in one of the earlier borings that crystalline rock like that of the Highlands occurred in the floor of the Moodna, under the deep cover of drift, but the adjacent borings showed only Hudson River sandstones and slates. Normally, the quartzite and limestone should lie between, but no such occurrences were discovered.

When the tunnel was driven, it was found that the stretch of crystalline rock was less than 200 ft. wide, that it was bounded on both sides by fault crush zones in which the rock was so weak that it required timbering, and that the formation immediately in contact on both sides is Hudson River shale. Undoubtedly, this crystalline strip is a remnant left stranded after two periods of faulting, and it is reasonably certain that it does not extend up from the granite basement, but instead is completely cut off below by these two faults. Doubtless, it would be possible, at somewhat greater depth, to pass beneath such a block as this and not encounter the granite.

The Great Thrust Fault.—At the margin of the Highlands where the tunnel line passes from the Hudson River slates and associated rocks to the Highlands granites and gneisses, the transition is sharp. A short distance north of the line, limestone may be seen along the contact within a few feet of the granite. A short distance south of the line, the limestone does not appear, but slate lies near the granite contact. North of the line, ½ mile or more, slate and granite can be seen in contact with a crush zone strongly developed and with the crystalline rock lying above the slate, the contact plane dipping at an angle of approximately 45° toward the Highlands beneath the granite. This represents an overthrust fault, the granite having been pushed up over the slate on a 45° plane.

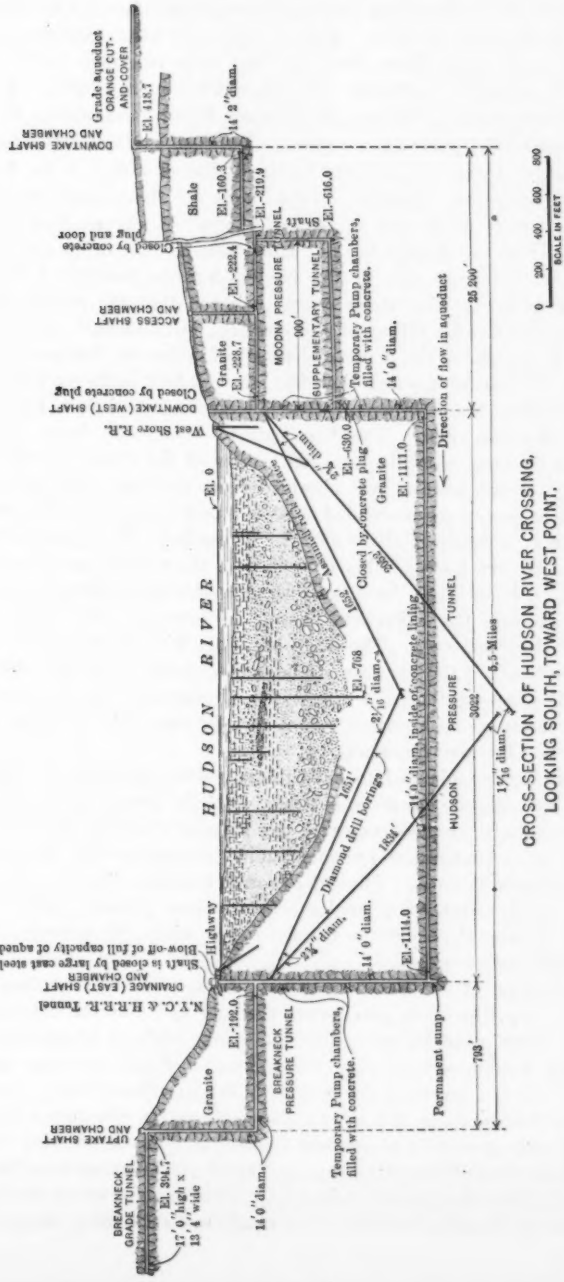
Borings placed so as to develop this structural condition at depth indicated the same fault on the tunnel line and with about the same angle of dip. When the tunnel was constructed, the fault was encountered as indicated. At that depth, however, about 200 ft. of limestone intensely crushed and rehealed, and so deformed by that process that its bedding could nowhere be determined, was encountered. On the west side of the limestone, the Hudson River formation continued and beyond the fault to the east only granite was found. The rock was badly crushed and slickensided and sheared in the vicinity of the granite, and a similar crush zone was developed at the contact between the Hudson River slates and the limestone 200 ft. farther west. Except at the two more pronounced crush zones referred to, the ground in the tunnel was good and sound for construction purposes and required no special or unusual attention. The great fault at this point which probably represents a displacement of 2 000 to 3 000 ft., and perhaps more, is crossed by the aqueduct without any structural difficulty. The cost of construction per linear foot of this tunnel was the lowest of any pressure tunnel on the aqueduct. It is evident from the success of the pressure tests and later operation that the depth of the tunnel is adequate and that no danger exists, due to structural weakness of the rock or to erosion notches, of sufficient consequence to interfere with the success of operation and permanence.

The Hudson River Crossing

A great number of factors were involved in fixing the location for crossing the Hudson River, some of which involve advantages of aqueduct location through the intervening country between the Catskills and the Highlands and others which apply in equal measure to its southerly extension toward the city. The vital and decisive factor, however, in selecting the exact location concerned the geological conditions at the crossing itself.

The possible places for crossing may be divided into two groups. One known as the "New Hamburg group" included all the crossings considered in the territory north of the Highlands where somewhat metamorphosed and badly deformed sedimentary rocks of the Cambro-Ordovician Age, including shales, slates, limestones, and quartzites, constitute the formations through which the tunnel would have to pass. The other known as the "Highlands group" represented the possible crossings within the limits of the Highlands. This Highlands region is characterized by crystalline rocks, including chiefly granites and gneisses of considerable variety, more or less disturbed by deformation, but, in general, comparatively sound. The New Hamburg group was explored sufficiently to determine that there would be a long pressure tunnel where limestone would be encountered under conditions encouraging to structural weakness. This ground north of the Highlands is known to be badly disturbed by faulting, and these conditions could not be avoided. It is a good general principle to avoid such objectionable weaknesses if it can be done, by locating in ground of more favorable quality. On this account, in large part, the Highlands group of crossings came into additional prominence. They avoid the weakened and deformed sedimentary series, except in so far as they would be encountered in crossing the Moodna Valley.

The several possible lines within the Highlands were further studied for individual differences. When the locations were studied in detail, it was found that the most southerly lines, especially the West Point line and any possible line farther south, would cross a large fault and crush zone under the Hudson River. It was also certain that all the southerly lines would cross much more varied rock types, and it was considered desirable to avoid this complication. Accordingly, the Storm King line at the extreme northerly margin of the Highlands was finally adopted as the best location. It was the opinion of the geologists that the whole section could be constructed at this point through granite of a single type from one side of the river to the other without crossing any major structural weakness; whereas an equally favorable combination of conditions could not be found elsewhere. It was recognized at the outset that the depth of the river gorge was not known and perhaps would be difficult to determine. This was also true of all other points. It was recognized that this particular location, by reason of glacial action, might be unusually deep. This seems to be fairly certain since the final explorations were made, but the argument in favor of the Storm King crossing was not offset by any of these considerations, because of the great advantage of locating this critical section of the aqueduct at a place where the major uncertainties of rock quality and structural weakness were reduced to a minimum.



CROSS-SECTION OF HUDSON RIVER CROSSING, LOOKING SOUTH, TOWARD WEST POINT.

Fig. 12.

Preliminary explorations were made on all the lines before the final choice of the Storm King location. Subsequently, extensive explorations were conducted on the Storm King line. Borings were put down in the river under extremely difficult conditions and to extraordinary depth. Fig. 12 is a geologic cross-section under the Hudson River, and shows the deep preglacial drift-filled gorge in the granite, the deep shaft and pressure tunnel of the Catskill Aqueduct, and the inclined diamond drill holes for exploring the rock before the question of the depth of the tunnel was decided. A depth of 768 ft. below the surface of the water was reached in the middle of the river by the deepest boring, which even at that depth did not reach the rock floor. At each side of the center, borings penetrated 500 ft. before reaching rock. At this stage it was believed that the results obtained had demonstrated the feasibility of the crossing, but sufficient data had not been secured to determine the depth and detail of design so that construction could proceed. It was determined, therefore, to sink test shafts on both sides of the river and from them to explore the underground conditions by inclined drill holes under the river. The first pair of holes, one from each side, was projected so as to reach beneath the center of the river at a depth of about 1 400 ft. Sound granite was obtained from the cores for the entire depth. After these holes were completed, a second pair was projected to cross beneath the river at a depth of about 950 ft. These holes likewise penetrated sound granite. On the basis of this information, the tunnel was placed at a depth of 1 100 ft. below river level, and was constructed without special difficulty.

The Hudson River Pressure Tunnel is located at the north edge of the granitic Highland area. The large building, Fig. 13, near river level marks the East Shaft, 1 100 ft. deep. The buried gorge is nearly 800 ft. deep to rock. The top of the hills marks the Cretaceous peneplain or base level, whereas the inner rock terrace, which may be noted about 100 ft. above water level, is the Tertiary peneplain.

It was at first planned to have the aqueduct approach the Hudson River section at hydraulic grade connecting with the tunnel under the river by a down-take shaft. Later studies of the Moodna crossing, however, led to the adoption of a continuous pressure tunnel connecting the Moodna, Hudson, and Breakneck Tunnels. Thus, the tunnels constituted one system at different levels. At the connecting angles between these pressure tunnels, structural weaknesses later developed in the solid rock when full pressure was applied, and repairs were required.

No faulting of any consequence was discovered in excavating the tunnel. Two or three slips were noted near the east end, but the original judgment that the river could be crossed at this point without penetrating any great structural weakness such as a crush zone or fault line was substantiated. In certain zones, however, the rock is under unrelieved strain, still preserved from the deformations of earlier geologic time. In the deeper workings, this strained rock at certain places had a tendency to scale off with violent explosions. Sometimes this scaling happened suddenly, breaking from the wall where previously the rock appeared to be sound. This scaling, which was known to the workmen as "popping rock", was encountered at several places and also in

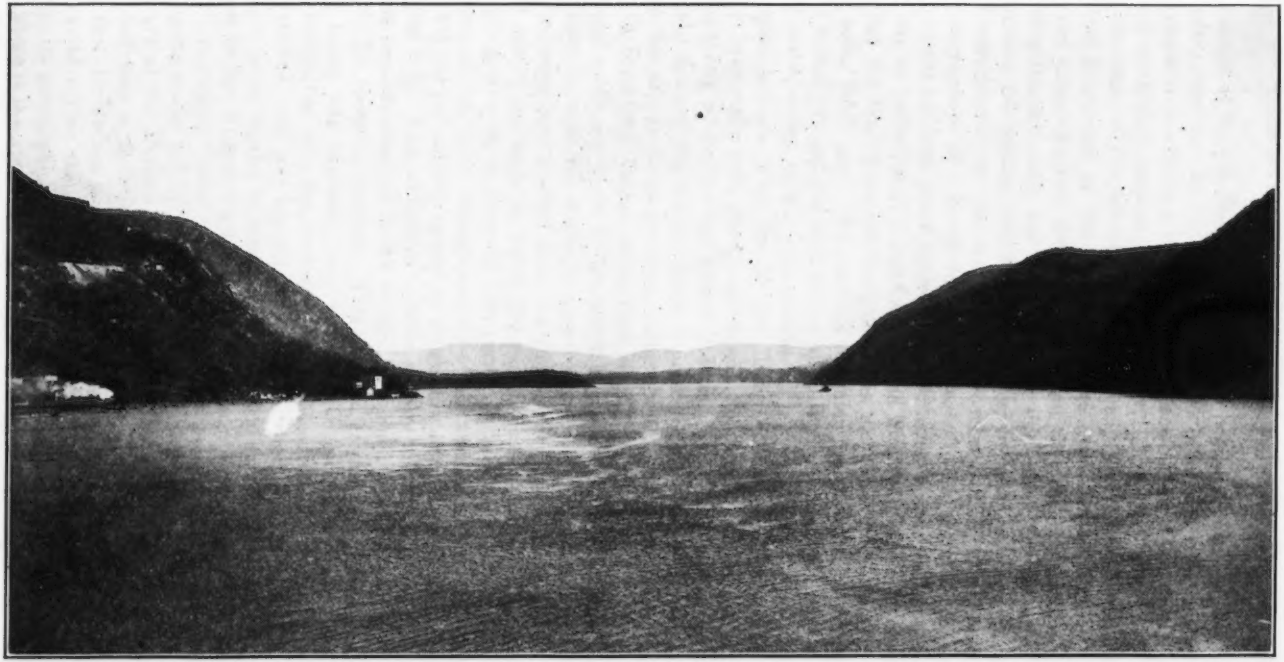


FIG. 13.—LOCATION OF HUDSON RIVER PRESSURE TUNNEL.

other tunnels of the aqueduct, but nowhere did it give as much concern as at the Hudson crossing. This condition of the rock is believed to have developed the weakness in the Hudson River end of the Moodna Tunnel after full pressure was established, making repairs necessary.

At one stage of the construction an unusually heavy flow of water from the joints penetrated by the tunnel near the east shaft required a large emergency pumping plant; the small equipment that had served successfully in sinking the east shaft was inadequate for the sudden flow of water just after turning the heading under the river. Fear of water had been expressed in the consideration of the tunnel project beneath the Hudson, and this sudden appearance of a heavy flow was thought to be ominous. However, after the joint had been closed by grouting, no further trouble was experienced. There was no indication of a direct connection with the Hudson River water itself and the large pumping equipment was not needed, the tunnel, as a whole, being fairly dry. The quantity of water for the 3 000 ft. of tunnel did not exceed 110 gal. per min. at any time after passing the inrush of the ground-waters stored in the joint system.

Moodna Repair.—When the Moodna-Hudson-Breakneck Siphon was subjected to hydrostatic tests, leakage developed. Subsequent examination showed that the lining of the Moodna Tunnel had ruptured at its junction with the down-take shaft of the Hudson River Tunnel and that water escaped to the surface through joints in the rock. There were, also, some cracks in the lining in Breakneck Tunnel at the river shaft.

Special study was given to the probable cause of this unexpected weakness where the rock had originally appeared to be sound and without water. The conclusion was reached that the rupture developed where the tunnel crossed one of the zones in which the rock is under strain. If penetrated at a little greater depth, it would have produced "popping rock", such as has been encountered in the deeper portions of the tunnel beneath the Hudson. Perhaps it was not unstable enough at the shallow depth to break without some additional help. This additional impetus appears to have been given by the bursting pressure supplied by the water-filled tunnel, when the rock was subjected to the operating pressure. This unstable ground then yielded and the tunnel lining was ruptured, thus allowing the escape of water into the joints connecting with the surface. On the Moodna side, therefore, where this disturbed zone was apparently several hundred feet in width, it was decided to make repairs by abandoning about 900 ft. of the Moodna Tunnel adjacent to the Hudson Tunnel and constructing a connecting step 400 ft. deeper. Rupturing under similar conditions was elsewhere corrected by caulking the cracks and by grouting.

Nature of the Hudson Gorge.—Because of its great depth, the Hudson Gorge has been a favorite topic for geologists and engineers. It is well known that the Hudson is a very ancient river and that at one time the Continent stood higher than it does at present. Under such conditions, the river cut its gorge, which can be followed by soundings nearly 100 miles out to sea where it is a veritable canyon from 3 000 to 4 000 ft. deep.

Knowledge of these conditions, together with the fact that hitherto no explorations had determined the exact depth of the gorge anywhere along its course below Albany, N. Y., gave especial uncertainty to the problem when the work first began. Later explorations have tended to show that although the Hudson River is very deep, more than 300 ft. at New York City, yet there are certain abnormal features of the gorge at the Storm King crossing that invite comment and require some additional explanation. The Hudson Gorge at Storm King is too deep and too wide to be wholly consistent with the cross-sections at other points. In its deepest part, below 300 ft., it is wider than at New York City, and it is known to greater depth. It is this fact, however, of greater width below the 300-ft. level, even though the rock is more substantial, that leads to the conclusion that the gorge has been "over-deepened" and enlarged by the glacial ice at the entrance to the Highlands. This is one of the clearest cases in the region of glacial over-deepening and enlargement. Probably nowhere else along its course is the Hudson so deep. The ice seems to have crowded into this notch between Storm King and Breakneck Mountains, gouging out the rock considerably deeper than it had been cut by the river itself. It appears, therefore, that the Storm King crossing may have required a somewhat deeper tunnel than would have been necessary at some other location, but it had other advantages quite offsetting this factor, and it was on these that the choice was based.

Special Features in the Highlands

The aqueduct through the Highlands comprises a succession of grade tunnels, cut-and-cover aqueduct, and steel pipe siphons. A variety of geological conditions was encountered, which were chiefly of scientific interest but which had some bearing on the original questions of location and design.

At one place in Foundry Brook Valley, a crush zone was encountered, which, at first, caused some concern, because the drill recovered only what appeared to be sand, but which proved later to be crushed and decayed material. This zone also carried much water, and one of the borings that penetrated it furnished an Artesian flow, which later was made the basis of a special claim for enhancement in the value of the land taken by the city. Another section of unusual character and local importance was in Garrison Tunnel, where a long stretch of residual decayed rock was encountered at the north end. It was assumed in the original survey that the rock which was a variety of gneiss, would be comparatively sound. In excavating the tunnel, however, a patch of pre-glacial residuary decay material was found at the north end beneath the drift, which the glacier had failed to remove. This gave trouble when excavated. A fault zone in the middle of the tunnel showed similar disintegrated material, but did not cause much trouble.

Sprout Brook Valley was thoroughly explored, but on account of the depth required for a tunnel, a steel pipe siphon was adopted as more economical, although a tunnel is feasible as far as geological conditions are concerned.

Fig. 14 is a view of the steel pipe siphon across Sprout Brook Valley. The valley is eroded on a narrow band of crystalline limestone with granite hills

on each side. The bottom of the valley is filled to a depth of 100 ft. with glacial drift. The opposite hill is penetrated by the aqueduct tunnel (Cat Hill Tunnel) for ½ mile.

In Peekskill Valley, the formations previously encountered on the north side of the Highlands were found to be closely folded, and affected by weathering and decay to a considerable depth. After exploration sufficient to map the occurrence, attitude, and condition of these formations, the steel pipe siphon was adopted as the method both for avoiding a deep pressure tunnel and some of the uncertainties of behavior of these rocks. Similar problems were encountered at numerous places where either small grade tunnels or short pressure tunnels were constructed.

Kensico Dam

At Valhalla, N. Y., a short distance north of White Plains, within about 20 miles of the city, it was decided to locate a storage reservoir. This is the site of the old Kensico Reservoir that was already a part of the New York City System, storing the waters of the Bronx River. However, in order to serve its purpose in the Catskill Project with a storage adequate for a three months' supply, it had to be greatly enlarged, and the geologic problems of this work were those incident to the greater importance of the high dam necessary for the increased storage.

At Kensico, the valley is comparatively narrow and deep, and its sides are rocky, although the bottom is filled with drift. It was known from the outcrops in the vicinity that the rock formation of the west side of the valley, at this point is Manhattan schist, that the east side is granite gneiss of considerable variety belonging to the so-called Fordham gneiss series, and that the Inwood limestone formation must lie in the bottom of the valley. It was evident, therefore, that the dam would stretch across the three important formations of the region, a fact which, in itself, would present no particular disadvantages unless unusual weaknesses, especially those relating to fault movements, should be found. Fig. 15 is a view of the foundation of the Kensico Dam. It is a narrow V-shaped gorge, at contact of Fordham gneiss (left) and Inwood limestone (right). The excavation is more than 100 ft. below the ground surface.

Explorations showed that the limestone, which was exposed in the bottom of the valley (Fig. 15), is 400 ft. thick and that, as a whole, it was in no poorer condition than the other formations. Perhaps, it is in better condition than the schist, because the latter is so much more broken.

The chief problems to be settled by detailed exploratory work were:

First.—The amount of drift cover and its character, especially its water-bearing quality as affecting construction.

Second.—The extent and distribution of weak zones or any broken condition of the rock.

Third.—The extent of weathering disintegration and decay.

Fourth.—The perviousness of the rock itself.

All these questions were determined within reasonable limits and were consistent with the earlier general estimate of conditions. After all the exploratory borings had been made and the facts bearing on these points had been summarized, it became evident that the more northerly lines had the soundest rock. The dam was placed, therefore, some distance north of the site of the existing old dam. On account of the discovery of considerable rock decay, especially at the junction between the gneiss and the limestone, and the presence of considerable jointing in the rock itself, particularly in the Manhattan schist, a design was adopted that provided for stripping and considerable excavation of the surface rock and a deep cut-off trench.

Of almost equal importance to the question of location and the quality of foundation is the problem of materials for construction. The dam required approximately 1 000 000 cu. yd. of structural material and the decision to use stone facing, if a suitable quality could be found near enough to be utilized economically, emphasized the problem of local supply. The basal gneiss series on the east side of the reservoir carries a large quantity of igneous intrusive rock of essentially granitic composition. This rock, the Yonkers gneiss, is of strikingly granular habit in most exposures and disintegrates to a sand under certain kinds of exposure. However, blocks of this stone used in the oldest buildings of the region are well preserved, and the stone shows high strength and a satisfactory resistance to the ordinary disintegration tests. Even the somewhat weathered material, considered by no means in its best condition, shows surprisingly good results under the tests. The color of the stone and its structural markings are considered to be very attractive. No other masonry structure on the Catskill work has been more successful from the standpoint of the material used.

The East View Concrete Disintegration.—From Kensico Reservoir, the aqueduct turns toward the city with cut-and-cover and short pressure and tunnel sections with comparatively little incident beyond the occasional uncertainties regarding the quality of the rock or the encountering of exceptional decay and disintegration. In Elmsford Tunnel north, a long stretch of excessive decay introduced the usual difficulties in construction. The only unusual feature, however, sufficiently different from those encountered in many other places, to warrant special attention, was in the East View Tunnel, about midway between East View and Kensico Reservoir. After this tunnel was completed and had stood for some time, it was discovered that, along a considerable stretch of roof and side-wall, the concrete lining was badly disintegrated. Patches were discovered that had softened completely and, in a few places, parts fell from the roof. It was found, however, that the trouble was not general, suggesting that there was some particular cause connected with the ground itself, rather than any fault of construction.

The aggregate used in the concrete in this tunnel included a great deal of limestone. At the time of construction, the suitability of limestone aggregate was not questioned, because this tunnel was put under comparatively little pressure and there was no thought of particular difficulty from the surrounding conditions. On investigation of the condition of the concrete, it was found that many of these limestone fragments had been completely de-



FIG. 14.—VIEW OF STEEL PIPE SIPHON ACROSS SPROUT BROOK VALLEY.



FIG. 15.—VIEW OF FOUNDATION OF KENSICO DAM.

stroyed. The experiments in this tunnel further emphasized the unsuitability of limestone for concrete aggregate under miscellaneous or uncertain conditions, especially if there is any probability of acid waters coming in contact with the concrete.

An investigation showed that the waters trickling into the tunnel in the part affected were charged with sulphates and had a destructive chemical action. Examination of rock that had been thrown out from the tunnel and from the borings made in this part of the ground showed large quantities of iron sulphide as a constituent. Blocks that had been exposed at the surface since construction were badly oxidized and showed abundant development of sulphate of iron which had been accompanied by the production of free sulphuric acid. A spoil bank of this kind of material had been thrown out on the surface almost directly over the most seriously affected part of the tunnel. From these facts it was concluded that the real cause of the trouble lay in the oxidization of the sulphide content of the rock, and the opening of an underground escape for the downward circulating water which, therefore, could leach these soluble matters out of the rock dump and reach the tunnel immediately below with the result of continuous destructive attack on the tunnel lining. Study of the methods of overcoming the trouble emphasized the difficulty of removing the cause. It was finally decided to reinforce the weakened part by lining it with different material. Accordingly, a brick lining was put in and all voids were filled with cement grout for a distance of about 2 000 ft.

The City Tunnel

The aqueduct reaches Hill View just outside the northern boundary of New York City at Elevation 295. At this point, an equalizing reservoir with a capacity of 900 000 000 gal. is located, which distributes directly to the city. After exhaustive study of alternative methods a deep pressure tunnel was adopted to extend from Hill View southward through the city to Brooklyn, N. Y. This was to become the trunk line from which water could be taken at many points. As finally worked out, the water was also carried to Staten Island and the Borough of Queens by means of steel pipe extensions. The adoption of a tunnel plan emphasized the matter of natural ground conditions and made it imperative to conduct extensive investigations. The manner in which these were co-ordinated with the other engineering studies forms an instructive illustration of the manner in which the Catskill Aqueduct project was handled.

A thorough preliminary engineering and geological study was made before any steps toward exploration were taken. Taking into account chiefly the engineering works already in place in New York City and the known difficulties due to city development, three trial lines were laid out by the engineers between the starting point and objective in Brooklyn. At this stage, the geologists were asked to make a study of the surface conditions along these three lines and the adjacent ground to a considerable distance on each side. It was then necessary to decide whether a shift of any one of the lines would materially improve these conditions. After careful study it was found that a

reasonably clear choice of a line could be made, but that certain unfavorable features could be avoided by shifting it somewhat from its original course. The choice was based on ground conditions, without sufficient knowledge of other factors to be certain that this new course would be practicable from an engineering standpoint. Therefore, it went for the second time to the Engineering Staff for consideration, with the result that minor modifications were suggested for re-study. These modifications were found to be generally consistent with the geological conditions, and, thus, with very little additional modification, the course that the aqueduct should take through the city was tentatively established.

These preliminary studies were sufficiently detailed to determine the points of uncertainty that would require active exploration, the nature of these uncertainties of condition, the purpose of these additional investigations, and something of the amount of exploratory work required. All the exploratory work was done by borings, but no conditions were discovered after the course was fixed that required material shift of the line.

General Questions.—In conducting these studies, the following general questions were kept actively in the foreground:

1.—What is the character of the rock along the projected conduit at the proposed depth?

2.—Is the depth proposed an appropriate one for a pressure tunnel of this character?

3.—Does the character of the rock vary sufficiently to affect the cost or the safety of the tunnel, and would anything be gained in either respect by a change of position or change of depth?

4.—What principles are most fundamental in weighing the evidence and in judging these points?

5.—What are the most questionable points along the proposed tunnel and where is exploratory work necessary to determine the local conditions fully enough to satisfy the needs of this project?

6.—What borings or other field investigations should be made to determine the facts about the uncertainties along the line and to make certain of the practicability of the pressure tunnel?

Geological Formations of New York City.—The variety of rock types within the city limits is somewhat greater than along the remainder of the aqueduct, but a knowledge of the three standard formations is sufficient to an understanding of the nature of all the local problems. Originally, these three formations lay one above the other to a very great thickness, but they are now folded together so closely that the parts exposed after long erosion stand almost on edge. On this account, the formations appear at the surface as long strips with a general northeast trend. The bottom and the top formations are more resistant to erosion and weathering than the one between them. As a result, the major topography of Manhattan and all the regions lying to the north of the East River and Long Island Sound have developed a parallel ridge topography consistent with this structure. Narrow valleys or the depressions lying parallel to this structural trend are underlain in most cases by the

softer rock (limestone) and the ridges are either one or the other of the two resistant members. These formations are: (1) the Manhattan schist at the top of the series; (2) Inwood limestone, the intermediate member; and (3) the Fordham gneiss, the basal member.

Originally, these formations were sediments varying from sandstones and limestones to simple mud shales. Now, after long metamorphism, these are gneisses, marbles, dolomites, and schists, each type presenting many variations in minor character and quality. The Manhattan schist, the youngest member, is essentially a mica schist. It is usually comparatively coarse and flaky in structure and inclined to be crumpled and somewhat bunchy. It is impregnated with igneous material which gives the rock a bunchy appearance or introduces stringers or dikes of pegmatite. Rock of this formation, because it is made up of very durable minerals and has a structural habit that makes it work in a massive form with reasonable satisfaction, has been considered suitable for tunneling.

The Inwood limestone is a coarsely crystalline dolomitic limestone. At depth, the rock is sound, but at the surface it crumbles readily under weathering influence and is comparatively easily eroded. Therefore, these belts of limestone are the lines of depression in the local topography. On the general principle that limestones of all grades are less stable where solution is possible than rocks made up of crystalline silicates, it was considered that long stretches of the Inwood limestone should be avoided if possible.

The Fordham gneiss is a black and white banded rock, made up chiefly of schistose and granitic bands. In some places much granite occurs in this formation, such as the "Yonkers gneiss". On the Brooklyn side, another similar occurrence is known as the "Ravenswood granodiorite". It is composed of durable material, but on account of its strongly banded structural habit, it tends to break into slabs, thus introducing a behavior somewhat more troublesome than that of the Manhattan schist.

Principles of Judgment.—At least one other structural feature must be considered. This is the crush zone or fault zone along which movement has taken place and along which rock is broken or weakened enough to encourage deeper decay and alteration than at other places. On the surface, these weakness lines are indicated by depressions. Relying on such knowledge of the importance of these conditions as was appreciated at the inception of this enterprise, the following principles were accepted as suitable guides, both in the matter of choice of location and exploration:

First.—It is in general desirable to avoid location in or near to a contact between two formations, as this is likely to be a zone of weakness underground.

Second.—Nearly all fault lines and crush zones are also lines of special weakness, and on this account it is desirable to avoid them in the manner indicated in Principle 1.

Third.—The behavior of the Inwood limestone is more uncertain than any of the other formations and, therefore, should be avoided as much as possible.

Fourth.—The simple igneous units such as the Yonkers gneissoid granite and the Ravenswood granodiorite are probably more uniform and consistent in their behavior under tunneling than any other formations, because of their simple crystalline character. For somewhat similar reasons the Fordham gneiss and the Manhattan schist may be considered comparatively satisfactory formations, although neither of them is as uniform in structure as the other two.

Fifth.—Along the lines of contact between the several rock formations and along crush zones, superficial decay and disintegration are much deeper than at other places. These places are practically the only points at which construction difficulties are likely to be encountered. They are also the chief points where the inflow of water and falling rock may be expected to be greater than usual.

Sixth.—The deepest erosion, marking pre-glacial stream channels, is to be expected also along the lines of contact and along the crush zones. The depths that these stream channels may reach cannot be determined without exploration. The positions of the most important spots, however, are indicated with reasonable accuracy so that exploratory work can be confined to definite places.

Seventh.—The overlying glacial drift is variable, but, for the most part, is very porous and water-bearing, especially where heavy cover is to be expected. This condition is a factor of importance in considering methods of construction.

Tunnel Location.—Basing judgment of the advantages and disadvantages of the trial lines on the principles just stated, it seemed reasonably clear that one of the three original alternative lines had decided advantages over the others in the proportion of better grade of rock and avoidance of questionable zones. One of the lines had an unnecessarily long stretch in the limestone formation. A second had a considerable length across a part of New York City where the underground formations are wholly obscured by drift. On the whole, the advantage seemed to lie with the trial line that was located farthest to the west, running from Hill View southward past Jerome Park Reservoir, crossing the Harlem River and running almost the entire length of Manhattan Island before crossing to Brooklyn.

In this connection, all the exploratory data available in the city were re-studied and a new geologic map was made of that part of the city within reach of the project. This map showed considerable differences from the standard geologic map of Folio 83 of the U. S. Geological Survey, some of the points being regarded as very radical changes. As a guide to location studies the contact lines between the several rock formations were re-drawn as accurately as possible, and special attention was given to lines of structural weakness and to depressions in the rock floor marking pre-glacial drainage. The course of the City Tunnel was based on these studies. Although this general location was made before additional exploratory work was undertaken, no new conditions were discovered of sufficient importance to require material modification. The question of depth, however, was not settled until the exploratory data from the borings were available.

Questionable Points.—By detailed studies of the ground along the course adopted for the City Tunnel, it became certain that, for the greater part of the distance of nearly 18 miles, the geological conditions were so fully determined and so satisfactory that no exploratory work would be necessary. Ordinary geological interpretation also reasonably established that long stretches of ground, even where direct observations of the bed-rock were not frequent, would not present conditions of sufficient uncertainty to warrant extensive investigation. At four points, however, it was realized that not only were the actual conditions unknown, but the evidence indicated greater probability of weakness than at other places. It was also expected that in places the rock floor would be deeply eroded and show structural weaknesses. These spots would probably control the depth of the tunnel and determine the difficulties of construction. It was advised, therefore, that these points should be thoroughly explored. These four places were:

1.—The vicinity of the Mosholu Parkway;
2.—The Harlem River crossing;
3.—The 125th Street cross-town depression and its extension southward to Central Park; and
4.—The lower East Side from the Bowery across the East River to Brooklyn.

Certain surface conditions suggested the probability of a cross-zone of weakness at Mosholu Parkway. This was not expected to be of as much importance as the others, because of the very substantial Yonkers gneiss type of rock. Explorations, however, showed satisfactory conditions at this point at the depth adopted.

At the Harlem River, the tunnel would cross from Fordham gneiss through the Inwood limestone to the Manhattan schist. This was known to be a line of depression and weakness, and although the geologic structure in general was well established, the exact conditions had to be determined on the basis of which the depth of the tunnel could be fixed. Explorations were conducted, therefore, in sufficient detail to determine the exact boundaries of the formations and their condition, particularly at the contacts.

The 125th Street cross-town depression is a prominent feature, and it was appreciated in the beginning that it indicated a structural weakness such as a crush or fault-line zone. Explorations at this point extended from 125th Street southward and showed the correctness of this interpretation. The earliest borings indicated not only that crush zone conditions existed, but that the rock floor was eroded to Elevation — 190. The 125th Street depression is one of the deepest erosion lines within the limits of the city, and is much more pronounced than the course of the East River. Exploratory work, carried southward in the vicinity of Morningside Park, as far as 110th Street, indicated a somewhat less prominent weakness extending parallel to the contact of the formations with comparatively deep erosion also along that line. It was partly to avoid the worst conditions of this zone that the tunnel was turned eastward at 106th Street instead of 110th Street. Explorations enabled an accurate map of the boundaries of the formations to be made and

the major structural features were determined. It was these findings that determined the course of the aqueduct and its depth.

The lower East Side of the East River Section also required extensive study, partly because of the greater extent of unknown or questionable ground, the entire lack of outcrops, and the dearth of earlier exploratory data. Nearly all the distance between the Bowery and the Brooklyn side of the East River required detailed exploration. It had always been assumed that the East River follows a zone of considerable weakness or a change of formations. The revised map threw considerable doubt on this assumption and the explorations proved also that the bed-rock condition of the East River south of 30th Street to Brooklyn Bridge is comparatively simple and sound. A little farther to the west, however, between Forsythe and Clinton Streets, lies a belt of ground in which great complexity of structure and very deep decay were discovered.

This decay and disintegration were found to extend extraordinarily deep. The rock floor reaches Elevation — 200 and is below Elevation — 100 for nearly the whole distance. In this section of the rock floor, the deepest part is more than 100 ft. deeper than at the East River itself. Doubtless, the pre-glacial drainage was through this territory across the lower East Side instead of along the present course of the East River. The present course is evidently controlled by the accumulations of obstructing glacial drift which filled the old channel more deeply than some of the adjacent ground, so that when post-glacial drainage was finally established, the new watercourses differed from the original ones.

Additional exploratory work was done at certain other points along the projected line as a check on interpretations and as a basis for contract specifications. Three of the four sections originally chosen for detailed investigation proved to be the critical and controlling ones in determining the nature of the most questionable conditions to be encountered and in deciding on the depth of the tunnel. No others were discovered, even during construction, of so great significance.

Depth of Tunnel.—The comparatively sound conditions of the whole northerly portion of the line to the Harlem River made it possible to construct the tunnel at a moderate depth, about Elevation — 150, to that point. The controlling feature of the next section, however, was the Harlem River, and as it showed a weakened condition of the rock to Elevation 160, it was decided to place the tunnel through this portion at a depth of about 350 ft. below tide level. Fig. 16 is a geological profile of the Harlem River Tunnel of the Catskill Aqueduct, showing the rock penetrated.

At the 125th Street depression, pre-glacial erosion extends to a depth of 200 ft. and the rock is somewhat broken. It became necessary, therefore, to place the tunnel still deeper. It was finally located at about Elevation — 350. As the quality of rock improved, and the floor was much shallower for many miles south, the design adopted allowed the tunnel to rise on an incline to Elevation — 250, and this level was maintained as far south as the vicinity of the complex conditions of the lower East Side, and the East River. The ground of the lower East Side, which showed pre-glacial erosion deeper than

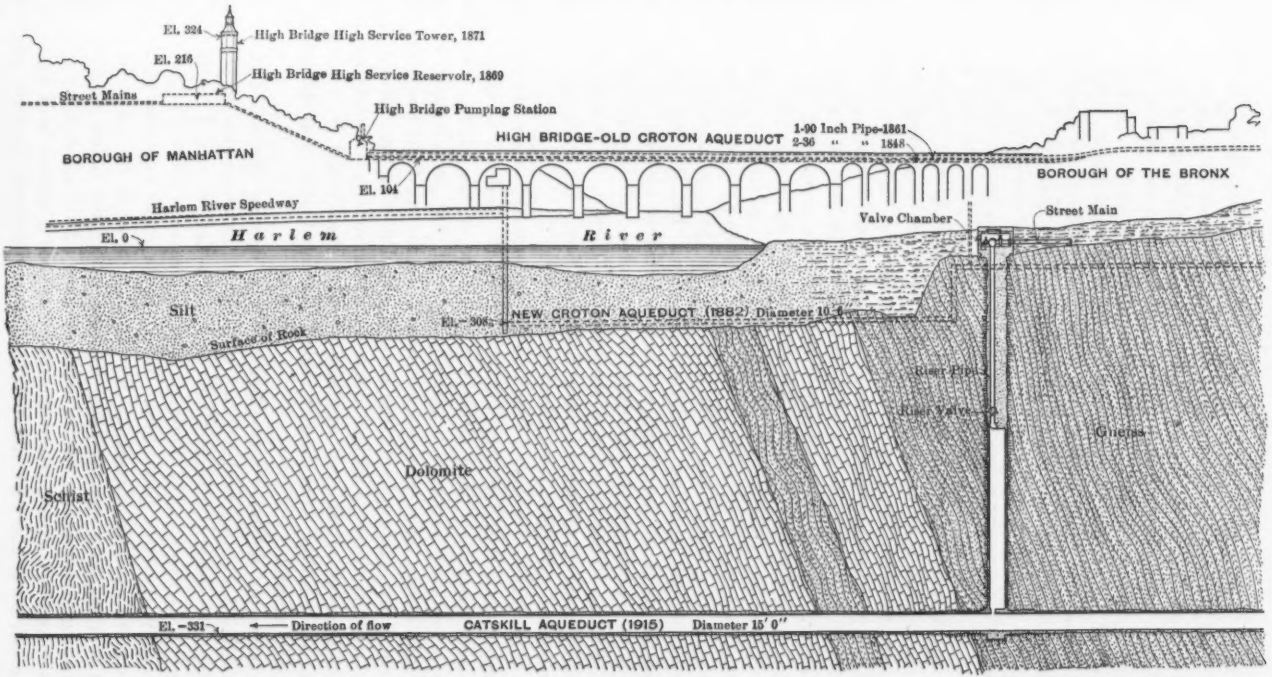

El. 324
High Bridge High Service Tower, 1871
El. 216
High Bridge High Service Reservoir, 1869
Street Mains
High Bridge Pumping Station
HIGH BRIDGE-OLD CROTON AQUEDUCT
1-90 Inch Pipe-1861
2-36 " " 1848
BOROUGH OF MANHATTAN
BOROUGH OF THE BRONX
El. 104
Harlem River Speedway
Valve Chamber
Street Main
El. 0
Harlem
River
Silt
El. -308
NEW CROTON AQUEDUCT (1882) Diameter 10' 6"
Surface of Rock
Riser Pipe
Riser Valve
Gneiss
Dolomite
Schist
El. -331
Direction of flow
CATSKILL AQUEDUCT (1915)
Diameter 15' 0"

FIG. 16.

Elevation — 200 and a weakened rock condition and decay to Elevation — 600, was regarded as so questionable in its behavior both for construction and permanence that it was determined to depress this section to Elevation — 700 in order to reach firm rock. On the Brooklyn side of the East River, the tunnel grade was raised again to Elevation — 250 to the terminal.

Fig. 17 shows the rock 700 ft. beneath Delancey Street. This rock is Inwood limestone, with complicated folding, crumpling, and faulting, and is cut through by a granite dike. The folding is characterized by remarkable crumpling varying from small plications of a few inches to great arches such as form the major ridges of the island and indicating a former deformation of the rock of considerable magnitude. Much trouble may be experienced in tunnels through folded rocks if they show marked fracturing, and careful attention must be given in such cases to the geological structure.

Observations on the results of construction and operation indicate that the deeper positions are the most successful. Athough Elevation — 700 would doubtless be excessive for the whole tunnel, the success of the work at the East River, together with difficulties that arose in certain of the shallower sections, have led to the conviction that the tunnel is nowhere placed too deep. There are weaknesses in all types of rock, and it is not the average condition but the weak spots which control the efficiency of the conduit under operating conditions. The behavior of the rock formations under construction and operating conditions has shown that the limestone formation at the depth of the tunnel has been a satisfactory type of rock, much more so than was expected. The difficulties expected from it were perhaps overstated, because of its behavior at the surface. There is no reason to believe that those parts of the tunnel where limestone forms the walls will be much less successful and permanent than other parts.

At some places, the banded Fordham gneiss formation has proved to be unsatisfactory as a rock for tunneling, because of its tendency to break out in slabs more extensively than either the limestone or the schist. The schist which in ordinary exposure is strongly foliated, has proved to be a satisfactory formation. Its crumpled nature and the fact that the schistose structure stands at high angles in the ground, coupled with the additional fact that the tunnel is at sufficient depth to avoid superficial weaknesses, has furnished almost ideal rock conditions. This rock could be broken very true to the required lines, and it seldom presented structural difficulties or dangerous ground.

Considerable difficulty was experienced in driving through those rocks that exhibited a strong tendency to break into thin slabs, especially when this condition was developed in rock that lay nearly flat or crossed the tunnel at a very slight angle. The Fordham gneiss presented these conditions and, occasionally, the other formations. Where crush zones were encountered not only was the ground dangerous for working because of the tendency of blocks to drop from the roof, but in some places the inflow of water and the disintegration of the rock on exposure gave additional construction difficulties. At 106th Street, where the tunnel crossed a known fault line with a wide crush zone, the rock did not present any unusual features when the



FIG. 17.—ROCK FORMATION 700 FT. BELOW DELANCEY STREET, NEW YORK CITY.

tunnel first penetrated it, except that it broke out in small pieces. Later, after exposure, the walls of the tunnel showed extensive disintegration and softening. At 106th Street, a heavy steel shell was placed for 110 ft. through the crush zone.

A worse condition, as far as excavation was concerned, was in the crush zones beneath the 125th Street depression, and somewhat similar conditions were encountered in the vicinity of 146th Street. At these places, the rock was broken more in blocks than at 106th Street and these blocks would drop out. In the vicinity of Jerome Park and elsewhere, some of the joints in the rock were water-bearing, one of which furnished for a time as much as 500 gal. per min. It was thought at first that this might connect with the reservoir, but the later behavior seemed to indicate that the chief water supply was local and connected only with the open joint system of the rock and the overlying soil.

At many points along the course of the tunnel, joint systems of considerable prominence were encountered. As a rule, these joints were not heavily water-bearing, but it was recognized that water would be lost through them, unless they could be closed. When the hydrostatic test was made, leakage developed near Madison Square, New York City. This section was repaired by lining the tunnel for some distance with sheet copper and by grouting the rock joints. No other sections of the tunnel have thus far developed weaknesses of this kind and its operation is considered to be successful.

Staten Island Development

Water from the Catskills is carried to Staten Island in steel pipes from Shaft 24. A cast-iron pipe was laid through the silt of New York Bay at The Narrows. A short tunnel in serpentine rock conducts the water to Silver Lake Reservoir on the high ground of Staten Island. The bed-rock is serpentine, much fractured at the surface, and of complex internal structure, but essentially substantial and water-tight at a moderate depth. The exposed parts are considerably broken and allow the circulation of water through the network of fractures or joints. This rock, however, is covered with drift, of which part is heavy substantial till and part assorted gravel and sand.

The site chosen for the reservoir is that of an original natural lake. It is, however, surrounded by glacial deposits, and explorations showed that these deposits, both at the southerly outlet and at the westerly side, are comparatively deep and variable in quality. The borings, however, indicated such a succession of interfingering or interlocking of porous and of impervious portions, that it was thought practicable to depend on this ground to hold water. The dike at the westerly outlet was constructed with a core-wall that reached to bed-rock, but did not extend the entire distance across the depression. The southerly outlet was so deep that no attempt was made to put a core-wall to bed-rock.

Some leakage developed under operating conditions; both to the west and to the south, the water travels in a roundabout way through the interlocking irregularly distributed porous parts of the drift, evading the more

substantial obstructing dikes and impervious material. Undoubtedly, greater reliance was placed on the quality of the drift and the water-tightness of the floor than conditions warranted.

Long Island

Originally, the development undertaken by the Board of Water Supply included a project for the development of additional supplies from Long Island sources. For many years the City of Brooklyn had secured part of its water from surface streams and wells many miles beyond the city limits. It was intended to develop the underground supply of Suffolk County and considerable progress was made before restrictive legislation caused the temporary abandonment of the Long Island project.

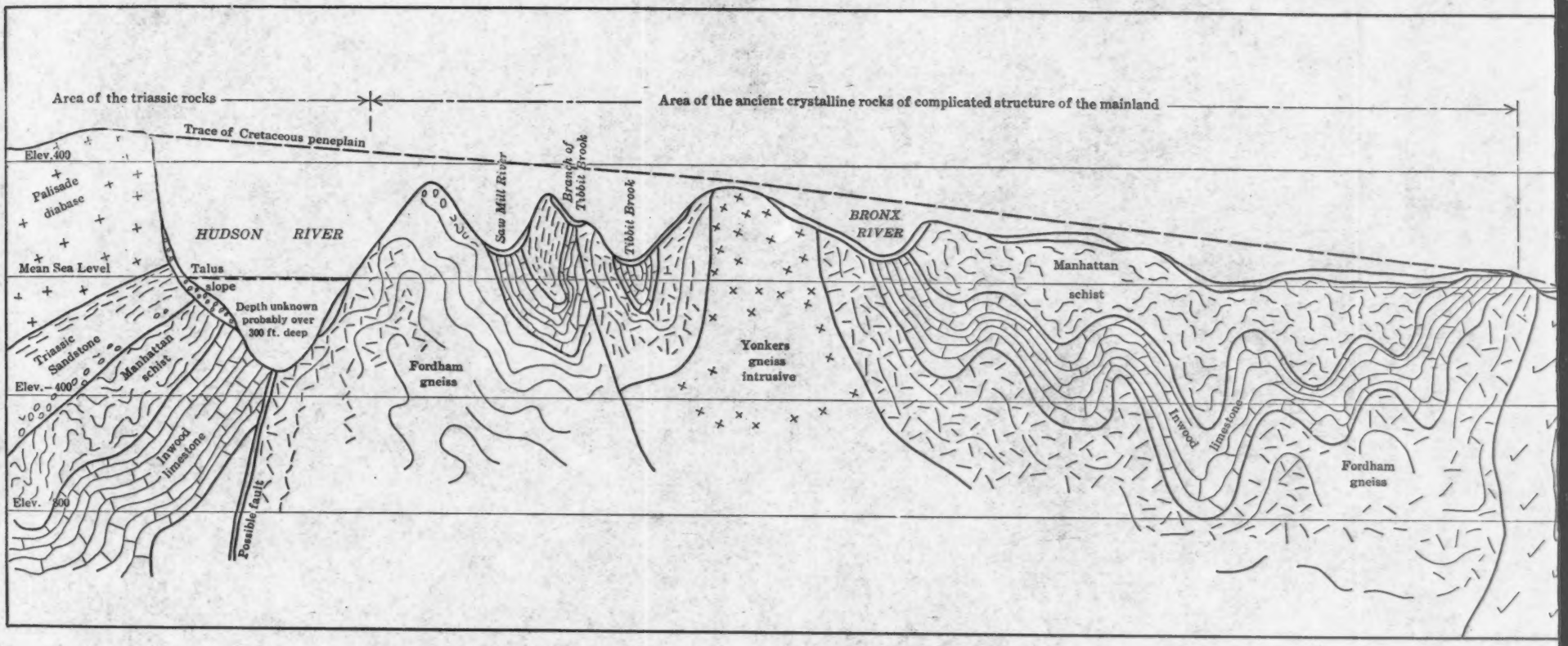
Underground supplies alone were sought, which, it was expected, might be increased by obstructing the run-off from some of the small streams in such a way as to give greater opportunity for the water to soak into the soil. The essentials of this project, therefore, have to do with the quantity and behavior of recovery of ordinary well waters and the underground conditions which govern them.

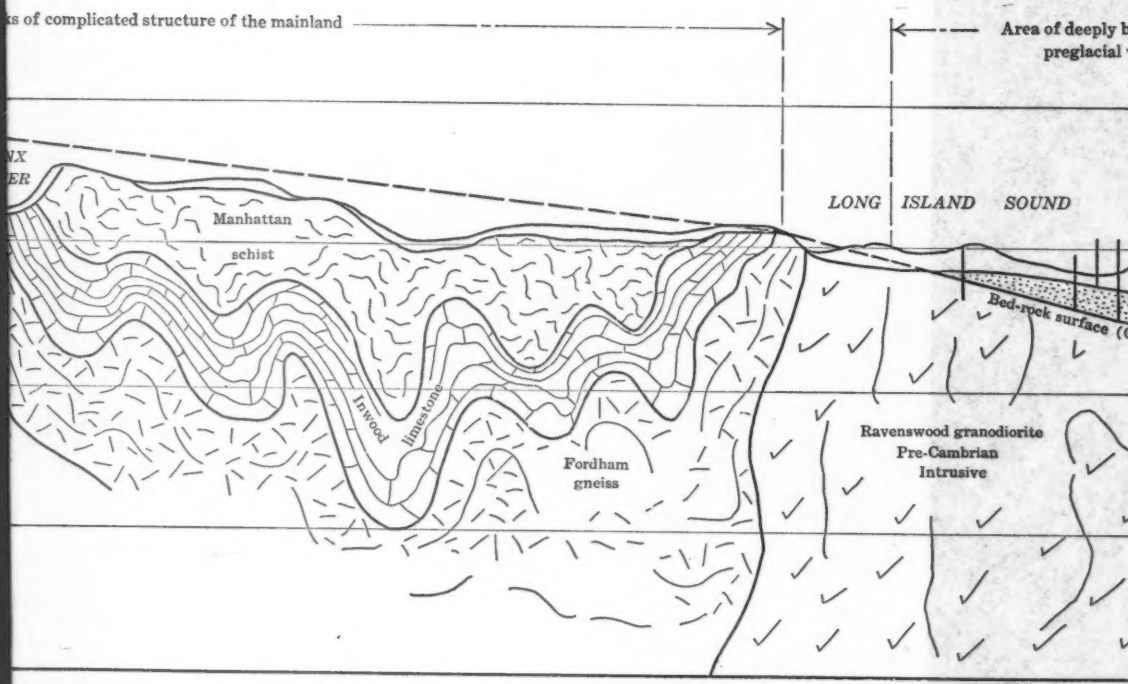
The coastal plain formation shown on Plate III, together with the glacial deposits on Long Island, overlie the ancient rocks and contain the porous members which are the main sources of water supply.

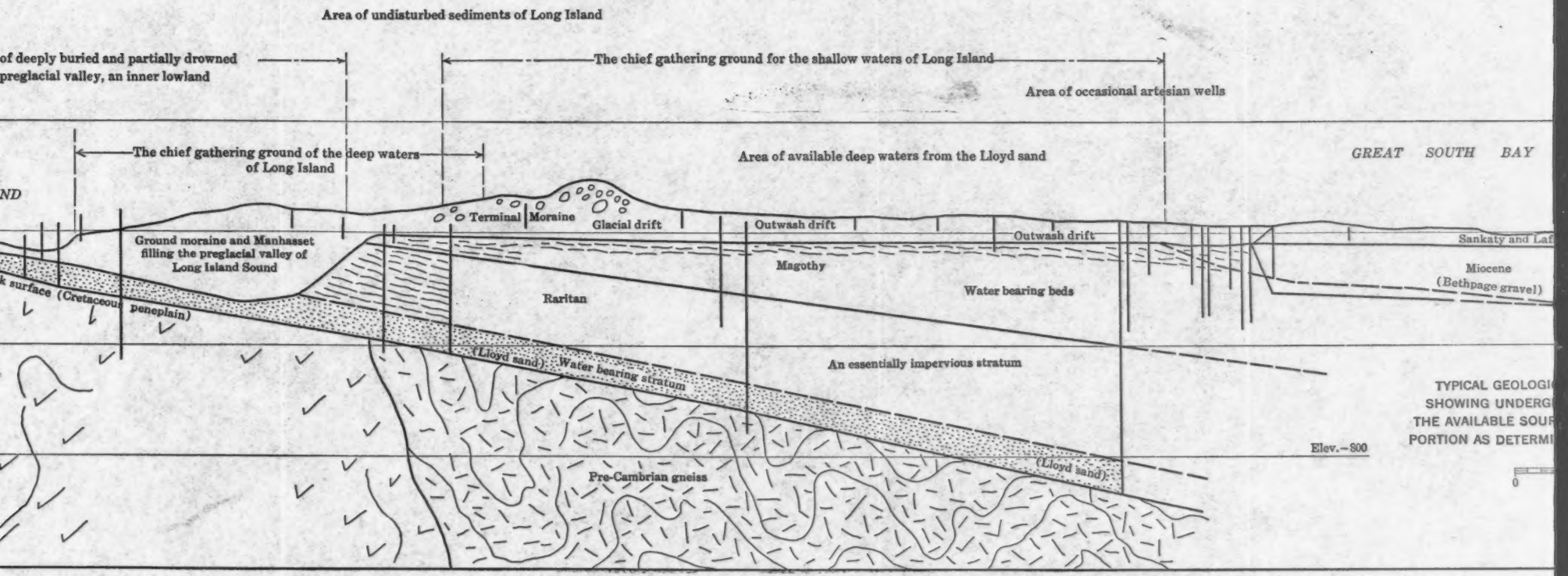
Immediately beneath the glacial drift of Long Island is a series of sedimentary sands and marls and clay dipping gently toward and beneath the sea. This whole series, in turn, lies on a uniform base of crystalline rock, the old eroded surface of which slopes in the same direction. None of the overlying strata extends farther inland than Long Island Sound and only the lowermost reach that far, because later erosion has beveled across these beds, so that if the glacial drift were scraped off, one could walk across the exposed layers one after another from Long Island Sound to the ocean. Thus, all of them are open to the entrance of percolating water along their entire landward extension.

All the layers are heavily covered, however, with glacial drift, including mixed morainic materials and an immense quantity of assorted gravel and sand. This glacial drift practically covers everything on Long Island, and its extreme porosity, which is its most striking characteristic, is also a most important factor in the problem of water supply, as ordinary rain water soaks into and through this drift cover and enters the underlying beds in great quantities. All these materials, both the glacial drift and the underlying sedimentary beds, are water-bearing, but because of their irregular surface and porosity, the depth to permanent water differs considerably.

Water obtained from these underground sources comes both from the drift and from the sedimentary beds at many different levels. Most of the supplies used are shallow and belong chiefly to the glacial drift. Deeper wells penetrate successive water-bearing sedimentary beds, some of which furnish a high grade and abundant supply. A particularly prominent source of high-grade water is known as the "Lloyd sand", which is one of the deepest layers,


Area of the triassic rocks
Area of the ancient crystalline rocks of complicated structure of the mainland
Trace of Cretaceous peneplain
Elev. 400
Palisade diabase
HUDSON RIVER
Mean Sea Level
Talus slope
Depth unknown probably over 300 ft. deep
Triassic Sandstone
Manhattan schist
Elev. – 400
Inwood limestone
Elev. 800
Possible fault
Fordham gneiss
Saw Mill River
Branch of Tibbit Brook
Tibbit Brook
Yonkers gneiss intrusive
BRONX RIVER
Manhattan schist
Inwood limestone
Fordham gneiss


s of complicated structure of the mainland
Area of deeply b
preglacial
NX
ER
LONG ISLAND SOUND
Manhattan
schist
Bed-rock surface (
Inwood limestone
Fordham
gneiss
Ravenswood granodiorite
Pre-Cambrian
Intrusive


Area of undisturbed sediments of Long Island
of deeply buried and partially drowned preglacial valley, an inner lowland
The chief gathering ground for the shallow waters of Long Island
Area of occasional artesian wells
The chief gathering ground of the deep waters of Long Island
Area of available deep waters from the Lloyd sand
GREAT SOUTH BAY
ND
Terminal Moraine
Glacial drift
Outwash drift
Outwash drift
Sankaty and Laf
Ground moraine and Manhasset filling the preglacial valley of Long Island Sound
Magothy
Miocene (Bethpage gravel)
k surface (Cretaceou peneplain)
Raritan
Water bearing beds
(Lloyd sand) Water bearing stratum
An essentially impervious stratum
TYPICAL GEOLOGI
SHOWING UNDERG
THE AVAILABLE SOU
PORTION AS DETERMI
Elev.—800
(Lloyd sand)
Pre-Cambrian gneiss
0

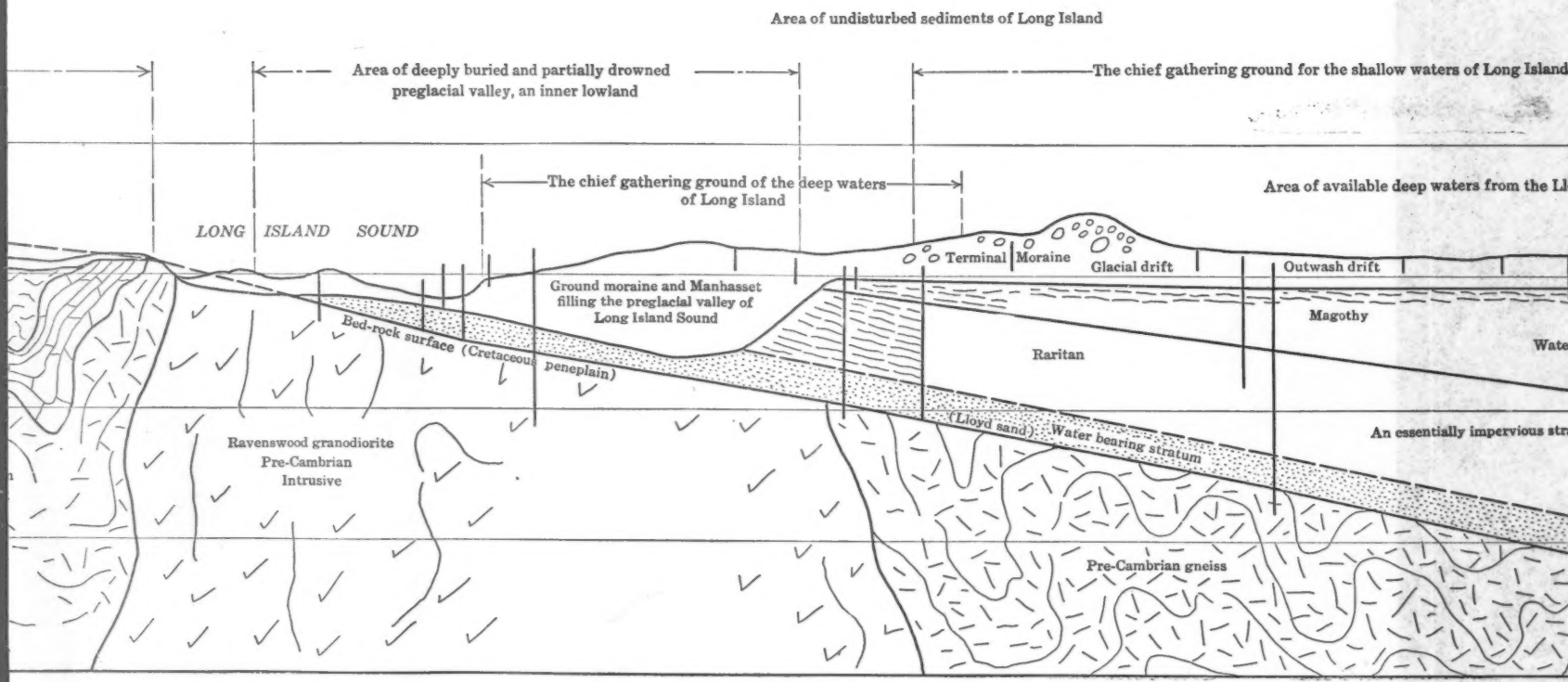
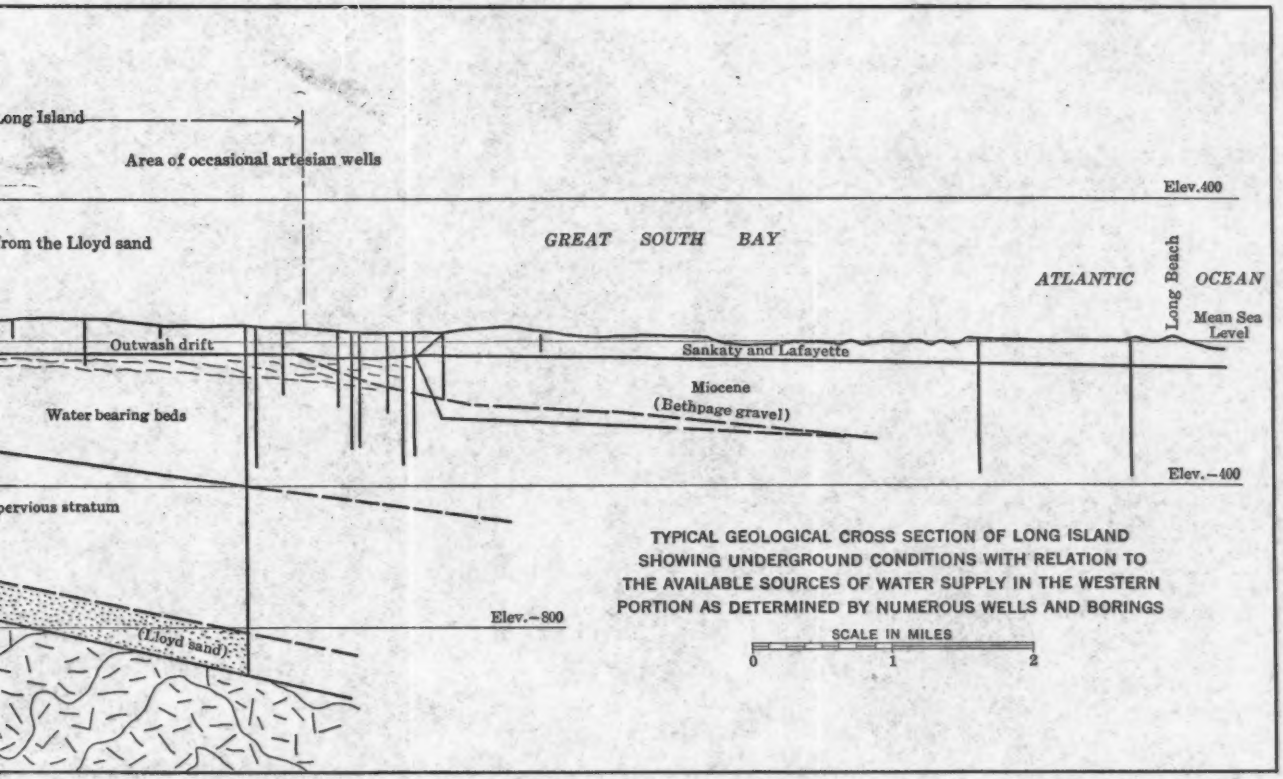

Area of undisturbed sediments of Long Island
Area of deeply buried and partially drowned preglacial valley, an inner lowland
The chief gathering ground for the shallow waters of Long Island
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Area of available deep waters from the LI
LONG ISLAND SOUND
Terminal Moraine
Glacial drift
Outwash drift
Ground moraine and Manhasset filling the preglacial valley of Long Island Sound
Bed-rock surface (Cretaceous peneplain)
Magothy
Raritan
Wate
(Lloyd sand): Water bearing stratum
An essentially impervious str
Ravenswood granodiorite Pre-Cambrian Intrusive
Pre-Cambrian gneiss

PLATE III.
TRANS., AM. SOC. CIV. ENGRS.
VOL. LXXXVI, No. 1509.
BERKEY AND SANBORN ON
ENGINEERING GEOLOGY,
CATSKILL WATER SUPPLY.



TYPICAL GEOLOGICAL CROSS SECTION OF LONG ISLAND SHOWING UNDERGROUND CONDITIONS WITH RELATION TO THE AVAILABLE SOURCES OF WATER SUPPLY IN THE WESTERN PORTION AS DETERMINED BY NUMEROUS WELLS AND BORINGS

SCALE IN MILES

0 1 2



and wells penetrating this stratum sometimes give Artesian flow. Many of the strata are too closely textured to permit ready movement of water. All the waters are of surface or rain-water origin. None of them comes from distant sources and the supply is limited to the quantity of such percolation. On both the north and south margins of the island, salt water encroaches on the fresh-water supply, the encroachment depending on the quantity of withdrawal.

The method of development, therefore, involves the sinking of wells into these water-bearing materials and developing this important supply. A large project would necessarily remove considerable of the underground water and would lower the ground-water level at the points of extraction. This feature was emphasized by the objectors to the plan, who finally caused temporary abandonment of the project. The ground-waters of Long Island are high grade and a large supply could easily be made available. It is near New York City, and development would cause a minimum of disturbance to other kinds of ownership and interest. Ultimately, as the city's demands grow, this source will be used on a much larger scale. It is the only near-by source of any consequence and also the only one capable of so large a development at small expense. The conditions governing this supply and the methods of recovery and control are certain to receive a great deal of attention in the near future.

Acknowledgments

The development of the Catskill Water Supply was begun in 1905 and was conducted by the Board of Water Supply, New York City. The Engineering Staff was under the direction of J. Waldo Smith, M. Am. Soc. C. E., Chief Engineer, until July, 1922; Thaddeus Merriman, M. Am. Soc. C. E., formerly Deputy Chief Engineer, is now Chief Engineer; John R. Freeman, Past-President, Am. Soc. C. E., William H. Burr, M. Am. Soc. C. E., and the late Frederic P. Stearns and Alfred Noble, Past-Presidents, Am. Soc. C. E., were the Consulting Engineers. Mr. J. Waldo Smith is now also Consulting Engineer. Professors W. O. Crosby and J. F. Kemp and Dr. Berkey were the Consulting Geologists. Acknowledgments are made by the writers to all who have contributed to this work.

DISCUSSION

ALFRED D. FLINN,* M. AM. SOC. C. E. (by letter).—The geographical extent of the Catskill Water System and its vital importance to a great community, added to the diverse geological features of the region, make this paper one of unusual interest. In passing, the writer would express his conviction, based on thirteen years' connection with the work described, and as many more years with other large projects, that the municipal authorities of the City of New York were fortunate in their geological advisers and in the fact that Chief Engineer J. Waldo Smith, Mr. Sanborn, and other engineers on the staff possessed knowledge and appreciation of geology. The paper is so complete in its description and so instructive in its correlation of engineering location, design, and construction, with geological exploration and interpretation, that the writer will simply emphasize a few general considerations.

That geological aid pays in constructional as well as in mining operations is believed to have been demonstrated. For greatest results in economy and security, it is essential that:

1.—The engineer and geologist should co-operate from the beginning of the preliminaries of the project.

2.—The geologist should be well informed on the broad geology of the region, the detailed structural geology of the location, the mineralogy of the rocks and the overburden of earth, and the present and ancient water circulation, or drainage.

3.—The geologist should have some comprehension of the nature and requirements of the structures and excavations making up the works of the project.

4.—The engineer should have some knowledge of geology and mineralogy.

5.—Even routine field explorations (soundings, test-pits, borings) should not be delegated to an assistant lacking understanding of the importance of each feature of such explorations. Every step requires the intimate supervision of an engineer possessed of knowledge of geology and keen appreciation of the importance of carefulness, close observation, and complete records, with clear, full headings and dates.

6.—The owner (or other person having control of the general policies) must have foresight and courage to spend enough money to obtain adequate information and make a thorough study of the facts gained.

7.—Full use should be made of the geological information and deductions in the design, location, and method of construction.

8.—All geological information, with suitable explanations and safeguards, should be made accessible to intending bidders, who should be required to examine such information while preparing their bids. Exhibits for bidders should include not only maps, sections, and statements, but also test-pits and materials removed from them, cores, and other samples from borings

* Secy., United Eng. Soc., and Director, Eng. Foundation, New York City.

(all of them, or as many as practicable), and every other physical object of contributory value, supplemented, if advantageous, with photographs.

9.—The geological information should also be accessible to the engineers and contractors during construction.

All these statements are somewhat obvious, but the results achieved on the Catskill Aqueduct work show many advantages which may be realized by the proper utilization of the skill of the geologist as an adjunct to good engineering.

GEORGE G. HONNESS,* M. AM. SOC. C. E. (by letter).—This paper is particularly interesting to those who were associated with the authors in conducting the work by which the geological information was secured. It also should be of value and interest to the Engineering Profession in general.

The part of the paper covering the work from the Ashokan Reservoir to the Atlantic Ocean is so complete, being based not only on the knowledge gained by borings, but also by the actual construction, that little can be added. The work on the Schoharie development is not so fully covered, as the authors did not have personal knowledge of the later investigations and the progress of construction. The writer, therefore, will try to add something gained by knowledge of the actual construction work.

Schoharie Reservoir and Gilboa Dam.—The sub-surface investigations for a site at which to construct a dam to form the Schoharie Reservoir extended from Pratt Rocks to Gilboa and located the pre-glacial gorge for a distance of six miles; everywhere the gorge below the floor of the old valley was filled with material which may be considered as generally impervious. The course of the old gorge was sinuous; near Devasego Falls, and, again, at the site of the Gilboa Dam, it makes a complete letter "S"; in the vicinity of Manorkill Falls, there was apparently an abrupt drop in the floor of the gorge and probably a "fall" existed in the pre-glacial stream. At Gilboa, the bottom of the pre-glacial gorge is at least 130 ft. lower than the post-glacial. The lowest part of the gorge is westerly from the limits of any engineering structure and is covered to a great depth by a very impervious clay. An investigation was conducted to determine the possibility of a filling of pervious character of wide extent in the bottom of the valley up stream from the dam. After making a number of special borings, it was concluded that if exposed rock adjacent to the dam was covered by earthy material deposited through water, no appreciable loss of water need be anticipated. Subsequently, in excavating the core-wall trench, it was concluded that the excavation was passing close to the south side of one of the steps of the pre-glacial gorge, and that some of the talus pushed off the sides of the old gorge by advancing ice had been penetrated. For a part of the distance, the core-wall extends to rock, and from where it leaves the rock to the known limits of this pervious material, an effort will be made to consolidate it by grouting.

The authors enumerate the "Geologic Conditions Affecting Dams", and it may be of interest to outline the methods used to determine the extent of the excavation for, and the treatment of, the foundation of the Gilboa Dam. The

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borings made for the preliminary investigation of the dam site, and also those made later for use in planning the structures, are shown on Fig. 5; many of these borings were deepened, those on the hillside being carried down to the level of the floor of the valley and those in the bed of the present stream to a depth of 100 ft.

Pressure tests were conducted to locate seams in the rock, and to determine their extent and their approximate water-carrying capacity when subjected to a pressure equal to that which would result from a height of 10 ft. of water over the crest of the dam. An apparatus similar to that used on other investigations made it possible to segregate each vertical foot of the diamond drill hole, to subject it to a pressure, and to make a volumetric determination of the quantity of water pumped. These tests showed that, in general, considerable leakage occurred in the upper part of the holes near the surface of the rock, both through vertical and horizontal seams; the number of seams and quantity of leakage decreased as the depth increased. When a test of one hole was in progress, observations were made in adjacent holes to determine the extent of leakage planes. No extensive leakage planes were detected; in some cases, however, responses were obtained in holes 400 ft. away. Seams were located both in the shales and in the sandstones, but they were more clearly defined in the sandstones.

In order to analyze and visualize the results of the test, a model was constructed, the outline of the dam and spillway being in plan. At the location of diamond drill holes, wooden circular rods were inserted and painted to indicate the various stratifications of rock. A blueprint showing the results of pressure tests was attached to the rods by thumb tacks at the proper position; cords leading from rod to rod showed the water circulation between holes and its direction. From a study of the results, a tentative depth for the general foundation of the dam and of the bottom of the cut-off excavation was fixed, and the depths to which to extend diamond drill borings for grouting were determined.

Fig. 18 shows a part of the profile which was issued to the field engineers as a guide to excavation. The line for excavation under Item No. 7 was tentative, because it indicated the extent of the excavation for the foundation of the dam, which was dependent on conditions disclosed as the work progressed. The line for excavation under Item No. 9 indicates the bottom of the cut-off trench, and is carried below the zone of seamy rock. Core borings inclined at an angle of $22\frac{1}{2}°$ with the vertical, were made for the purpose of intersecting heading joints and planes and thereby causing more effective circulation of the grout. The diamond drill holes were extended to a depth below which the pressure test showed only slight seams which carried off very little water.

The grouting is not done until masonry has been placed in the cut-off trench and to a substantial depth over the entire foundation of the dam. Steel pipes are caulked into the diamond drill holes and carried up with the masonry. Fig. 18 shows much of the information on which conclusions were based.

Perhaps the most interesting geological feature to the general public disclosed by the work in the vicinity of Gilboa was the finding of fossil trees of the Devonian period, said to be the oldest flora of the earth's history. The first fossils were discovered several years ago when a section of State highway was under construction. Dr. John M. Clarke, of the State Museum of the State Department of Education, asked that a lookout be kept for specimens as the work progressed. Before the construction had progressed far enough to disclose any fossils, representatives of his Department discovered some very good specimens at Manorkill Falls at about Elevation 1120. As the excavation for the dam progressed, other specimens were found at Elevation 1020 and at the quarry for facing stone at Elevation 960. In each instance, they were bedded on a thin layer of dark shale which represented what was left of a black muck swamp in which they originally grew. These fossils are said by the geologists to be 100 000 000 to 300 000 000 years old and are the only specimens yet found. A large collection has been placed in the State Museum at Albany, N. Y. Specimens are also in possession of the Harvard University Museum and the American Museum of Natural History in New York City.

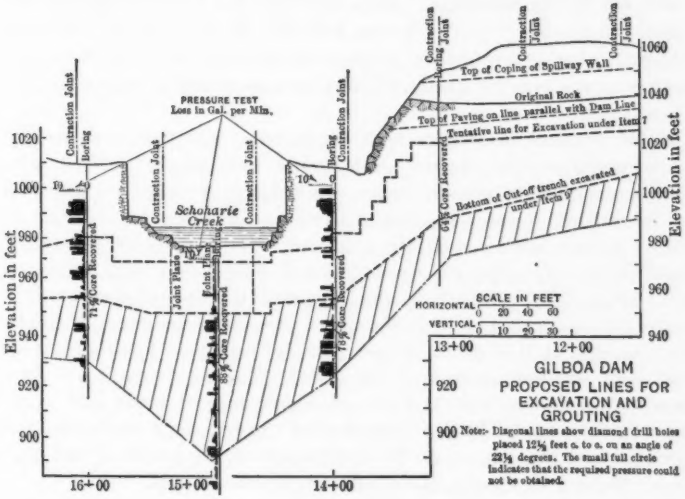


FIG. 18.

Shandaken Tunnel.—The preliminary investigation of sub-surface conditions disclosed that each small valley crossed by the line of the tunnel had its own particular gorge, and the investigation was sufficient to ascertain whether the pre-glacial gorges were deep enough to reach the grade of the tunnel, which occurred in only one case.

If the tunnel had been constructed on the hydraulic gradient at the Bearkill Gorge it would have penetrated the filling of the gorge, a disconcerting occurrence in tunnel construction. To avoid the possibility of this, the tunnel

was depressed so that there would apparently be a rock roof 100 ft. thick at the Bearkill Gorge. The Shandaken Tunnel, therefore, acts as a pressure tunnel for about 3½ miles of its length.

It was the judgment of all who had studied the borings that comparatively little support would be required and that where it was necessary it would be temporary, merely to protect the workers from small fragments spalling off. The condition encountered required that about 50% of the tunnel should be supported by either temporary or permanent timbering. The exposure of the red shales and sandstone to the atmosphere caused masses of sufficient size to fall, and secure protection was necessary. In the gray sandstone, "popping rock" which was observed at Hudson River shafts and other parts of the aqueduct, occurred, particularly in the north tunnel from Shaft 5. This rock appeared to be of the best texture and sound, but it required permanent support. After the heading and the tunnel for a distance of 100 to 200 ft. from the face of the heading had been excavated, it was common to hear a loud report and then a large slab of rock would be observed to loosen. These occurrences caused a panicky feeling among the men, as frequently large pieces of rock fell shortly afterward. After the timbering had been erected and dry-packed, the "popping" was no longer noticed. With the strain removed, it is problematical whether further working of the rock would have taken place, but the chances were too great, and support was carried as close to the heading as practicable.

A few crushed zones have been encountered, but they have been passed without much trouble. In the south tunnel, from Shaft 3, a horizontal clay seam, surmounted by a badly broken up bedding of shales, was encountered, and the inflow of water reached a maximum of 170 gal. per min. As the heading was advanced and heavily timbered, several crushed zones were passed where the stratification was tipped up and very blocky and, finally, a mud seam, varying from 5 to 7 ft. thick, crossed the line of the tunnel. This was possibly a decayed rock or, perhaps, a pre-glacial residuary decayed material, although, at that depth, it seems hardly possible. For the next 100 ft., three badly crushed zones were penetrated. The rock was broken by cross-joints and shrinkage planes, many of which dripped water and were filled with a bluish clay. The disintegration of the red shaly sandstones and the filtering in of the clays give the appearance of a mixture of red and gray shales. Beyond that point, the conditions become normal.

In the north tunnel, from Shaft 4, about 3 600 ft. from the place just described, a crushed zone which gave much trouble, was penetrated.

In the north tunnel, from Shaft 5, a small crushed zone and slight faulting were found, and, at one place, a badly disintegrated rock. Gas in small quantities entered in a number of places, and Plutonic water having a solid content three times that of the water of the Atlantic Ocean and one-half that of the water of the Dead Sea has been found, that was undoubtedly entrapped and held in cavities at the time the rock was being formed.

At present, only 4 000 ft. of the 18.1-mile tunnel remains to be excavated, and it is unlikely that more serious troubles than those described will arise.

At the southerly end, a short stretch of tunnel in earth is to be excavated, that presents many of the difficulties of soft ground work. The adoption of careful soft ground methods by an expert on that kind of work gives assurance that it will be successfully completed.

The tunnel has been remarkably dry, the maximum quantity of water pumped at any one time from one shaft being about 170 gal. per min.

The authors state in a number of places that, generally, the strata in the Catskill region "dip to the west at a slight angle". It has been found that the dip is from 1° 30′ to 2° in a southerly direction, consequently, that part of Plate II, covering the Shandaken Tunnel, should be corrected to agree with the determination given.

The character of the rock and the superior equipment and organization of the contractor led to remarkable progress in excavation. With twelve headings being worked, more than one-half of which required timbering, a progress of 5 593 ft. per month was made. The average weekly progress (elapsed time, which includes time lost in re-organization) is 62 ft. of completed tunnel per point of attack, a much higher rate than that maintained on the tunnels of the Catskill Aqueduct. At the shaft, maintaining the best progress, the average was 90 ft. per week.

LAZARUS WHITE,* M. AM. SOC. C. E.—The speaker's knowledge of geology gained from Professor Kemp at Columbia University, was valuable, and was used for the first time on this work. It was a great help in anticipating underground conditions, which is essential in order to locate an aqueduct or dam in a country like that south of the Catskills. The superficial aspect of that country does not indicate the underground structure, which is the result of past geological conditions, and about which a competent geologist can tell a great deal. He cannot tell all about it, and that is where the danger lies. The engineer can co-operate with the geologist, but he must adhere to an independent judgment of the conditions based, not on a geological theory or accepted statements, but on borings. To do otherwise invites the risk of serious error. For instance, on the Rondout Siphon, with which the speaker is familiar, the geological predictions were good, but it would have been dangerous to have located the aqueduct on the basis of those predictions alone, because one of the gorges that the geologists had predicted would be deep, was shallow, and another that they had predicted would be shallow, was deep; if the depth of the tunnel had been determined in accordance with the predictions, a deep gorge would have been penetrated, which would have caused serious difficulties. However, these discrepancies were discovered by diamond drill borings long before the final profile was adopted. It does illustrate, however, that too great a duty should not be placed on the geologist. The location of the Loetschburg Tunnel was left to the geologists, and no borings were made. The result was that the tunnel penetrated a buried gorge, with disastrous results.

There were places on the Rondout Siphon at which a change of profile would have made a great difference in the amount of tunneling in Hudson

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River shale and Shawangunk grit. To determine the cost of tunneling in each of these rocks, two experimental tunnels, side by side, were driven 150 ft. in them. This work furnished valuable information, especially on the method of timbering the shale, and resulted in a large saving, later, in the cost of driving the main tunnel.

In the Rondout Siphon, a particularly bad condition was found in the limestones, and there was no method of locating the profile to avoid them. One of the geologists predicted very closely the caves that might be penetrated in driving that tunnel; however, those caves were below the gorges. The water circulated down and rose again under pressure, the conditions being similar to those which the same geologist had observed in Missouri.

In the speaker's opinion, this paper is a valuable one for engineers, and can be used to great advantage in schools to co-relate the study of geology with that of engineering.

WALTER E. SPEAR,* M. AM. SOC. C. E.—The authors are to be congratulated on their clear exposition of the many problems in engineering geology that arose in the construction of the Catskill Water-Works. Perhaps in no previous enterprise of the same magnitude has there been equal co-operation between the engineers and the geologists.

The Hudson River is the one large stream within the State of New York, that reaches the sea without crossing the boundaries of another State. It has been possible, therefore, with the consent of the State, to appropriate its waters for New York City, and no attempt has been made to acquire any source of supply outside the State. The one large tributary of the Hudson River in the vicinity of New York City—the Croton River—was developed as the first large source of supply. It was necessary, however, to obtain the next supply from a more remote tributary of the Hudson River, the Esopus Creek in the Catskill Mountains. The difficulties of conveying the Catskill Mountain supply to the city were those of constructing an aqueduct across the geological formations which, in this vicinity, have a general northeast-southwest trend. If the Housatonic River, which was suggested as a source of supply in 1900, or even the more remote Connecticut River, was in New York State, instead of in Connecticut, the transportation in a direction parallel with the strike of the geological formation would have been simpler than bringing the Catskill supply to New York City.

It can now be estimated that the probable date of exhaustion of the available water supplies for New York City is not very distant. Preliminary investigations, anticipating the acquisition of new sources of supply, have already been started, in which Dr. Berkey has had a part. Now, as before, no works outside the State of New York may be considered, and the problems confronting the engineers are the same as those of the Catskill works, namely, constructing reservoirs and building a large aqueduct that will pierce the mountains and cross the deep valleys lying between the city and any large source of supply that may be made available. A new conduit will probably be built in tunnel to a larger extent than the Catskill Aqueduct. However, the

* Dept. Engr., Board of Water Supply, New York City.

experience gained on the Catskill Aqueduct in tunneling, mostly through the same formations as will be encountered on a new aqueduct, gives assurance that, with the same care and thoroughness in making the preliminary investigations, no serious difficulty need be feared.

Something may be added to the statements made by the authors in regard to the investigations on Long Island, looking to the development of an emergency ground-water supply which could be made available in advance of the delivery of the Catskill supply. In 1907 and 1908, in addition to the mapping of the water-table in Suffolk County, the gauging of the streams, and the topographical surveys, the Board of Water Supply made a number of deep borings on the south shore of Long Island along the line of development of the proposed ground-water supply. One of these borings, having a 12-in. stove-pipe casing, extended to a depth of 900 ft.

These borings furnished much additional information on the geology of Long Island and led to the preparation of a plan for pumping a supply of 250 000 000 gal. of water per day from the "yellow gravels" of Southern Suffolk County. Professor W. O. Crosby, the Consulting Geologist on this work, made an exhaustive study of all the available data on the geology of Long Island, which, previous to the work of Messrs. Crosby, Veatch, Bowman, and others in 1903, was, as a whole, a comparatively unknown territory. In his report of 1910, which is as yet unpublished, Professor Crosby confirmed the conclusions, previously reached by the engineers, that, in Suffolk County, the only available water horizon from which to obtain a large supply was that in the "yellow gravels", which he classified as the Cohansey and Lafayette of the Pliocene, the Pensauken of the Epipliocene, and the glacial outwash of the Pleistocene, and which cover Southern Long Island to a depth of 50 to 200 ft.

He pointed out that the Bethpage gravel of the Miocene, an important horizon in Southwestern Long Island, represented local beds deposited by mainland rivers or, perhaps, by glacial streams, and that these gravels did not exist in Suffolk County, except possibly as offshore deposits and except for a small body of gravel, under the Bethpage clay beds, which has no water-supply importance.

Of other water horizons on Long Island, Professor Crosby's report shows that many successful wells, generally of small capacity, are found in the Magothy of the Cretaceous series within a belt along the north shore in Queens, Nassau, and Suffolk Counties, from Flushing to Port Jefferson, where the water-bearing gravels of this formation lie at elevations referred to sea level of + 70 to — 200 ft., and at a few localities on the south shore in Nassau County, where this gravel was found about 500 ft. below sea level. It was Professor Crosby's belief that the Magothy in Southern Suffolk County contains no gravel, and if it does exist in localities thus far unexplored, it lies beneath such thick beds of impervious sand and clay as to be cut off from the source of supply in the rains that fall on the surface of Long Island.

The lowest water horizon on Long Island, the Lloyd sand (Raritan), also of the Cretaceous series, lies still deeper than the Magothy gravel and not far

above the bed-rock. It is a more important horizon than the Magothy gravel and furnishes a large quantity of water at moderate depths of 200 to 400 ft. below sea level on the north shore of the Island, in Queens and Nassau Counties, from College Point to Huntington, and within a zone lying north of that in which wells in the Magothy occur. A few wells have been driven to the Lloyd sand in Southern Queens, where it is found at depths of 500 to 700 ft. below sea level. Professor Crosby pointed out that this horizon derives its supply, as does the Magothy gravel, from the Manhasset and outwash sands and gravels that fill the buried valley of Long Island Sound. If Lloyd sand occurs in Southern Suffolk County, it lies at still greater depths than in Southwestern Long Island, and it is more effectively cut off than the Magothy from the surface sources of fresh-water supply and is probably filled with salt water.

This work of the Board of Water Supply on Long Island is deserving of wider publicity. The knowledge gained there in regard to the deeper water horizons should prevent some unnecessary explorations in the future and the drilling of many useless wells.

ARTHUR S. TUTTLE,* M. AM. SOC. C. E.—The authors have omitted all reference to the work performed by them, a few years ago, in assembling, for convenient examination, records and specimens of all borings in New York City and vicinity. The nucleus of this collection consisted of the boring record made in connection with the development of the Catskill Water Supply, but it was reinforced by all other available data that were reliable. The result has been that a vast amount of data has been collected which should be of enormous value in the development of plans for engineering work in this locality. This collection of data is housed on the thirty-fourth floor of the Municipal Building, and it is in such form that any information desired can be readily obtained. It is hoped to add to this collection by securing similar data from work now in progress and from work to be undertaken in the future. The fact that this collection has been made and is under the care of the Chief Engineer of the Board of Estimate and Apportionment is probably not generally known, and the opportunity is taken, at this time, not only to advise that any one desiring such information will know where it can be secured, but also to invite any contributions to the collection, which are now or may be later available.

LOUIS L. TRIBUS,† M. AM. SOC. C. E.—The speaker has in mind two cases that might be of interest in showing the advantages of co-operation between the geologist and the engineer. He was sent to take charge of a reservoir that had been previously under construction for about a year. The earthern embankments were partly built and the bottom had been partly stripped. The plans contemplated a clay puddle, 2 ft. thick, as the bottom lining. Six or eight test pits had been dug, which showed a fair quality of loam and clay, with no rock appearing. However, when the bottom was fully excavated, it was found that about three-fourths of it was underlaid with limestone rock

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† Cons. Engr. (Tribus & Massa), New York City.

having pockets 6 to 8 ft. deep. If a local geologist had been consulted or reasonable borings had been made, many thousands of dollars would have been saved. Concrete had to be used instead of clay puddle for the bottom lining.

Another case came into the speaker's practice a few years ago: A rather important dam had been partly constructed. When the bottom was first stripped, it appeared to be solid, but when the contractors commenced to excavate footings for the concrete, pockets were discovered large enough to hold small buildings, and washouts aided further in contributing knowledge. Owing to an almost complete absence of borings and lack of geological information, the owners spent several hundred thousands, and possibly, directly and indirectly, millions of dollars, to make good the deficiency.

Both the experiences mentioned are helpful in emphasizing the advantage of first obtaining geological data.

E. G. HAINES,* M. AM. SOC. C. E.—This paper differs from those usually presented before the Society. Primarily, it describes the work required on a rather extensive underground survey, and the information obtained therefrom. Such underground surveys are made for other engineering works, but they seldom cover as wide an area, nor are the investigations carried to as great a depth. In this case, the investigations were made largely by borings, often several hundred feet in depth (in some cases, as much as 1 000 ft.), whereas, for other engineering structures, a limited penetration into bed-rock is usually sufficient.

The paper presents the results from such borings verified by observations from the actual construction. Few engineers are familiar with the geological history, or even the geological structure, of the section on which they are employed. The speaker's attention has been called many times to that fact in connection with borings. He was once asked to examine a drawing called a boring sheet, which covered quite an extensive area in New York City. At the bottom of a number of holes, "white sandstone" had been recorded and this immediately aroused his suspicions, as he believed it to be limestone. The fact that the same material was shown at the bottom of a number of holes some distance apart indicated a general vein, and not boulders. It is difficult to understand how those who made the borings or recorded them on the sheet could have expected to find a vein of white sandstone enfolded, and in contact with Manhattan schist, which is a metamorphic rock folded and banded, and jointed in every direction, and eroded by glacial action. In this case, there was no excuse for an erroneous classification of the material, as a chemical and physical laboratory was available. The material in question was Dolomite, locally known as Inwood limestone and farther up the Hudson as Tuckahoe marble.

It may be held by some that this error in classification was of little importance, as the material was actually rock. That will not be conceded; the borings were made, not only for the information of the designers of an important structure, but also for contractors expected to bid on the work. At certain localities, the outcrop of Inwood limestone is disintegrated to such an extent that it is used as a building sand, whereas, in underground veins,

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subjected to water and the acids in the soils, it is sometimes decomposed and resembles clay. Between these two extremes lies the more solid rock, which somewhat resembles sandstone.

The point the speaker wishes particularly to stress is this: If borings are made, the boring samples should be classified by some one competent to interpret them properly. This is not always the case. Many borings have been made in New York City and vicinity, the samples of which are incorrectly classified, particularly wash-boring samples, of which part of the finer material is lost in the process of obtaining the sample.

X. Henry Goodnough,* M. Am. Soc. C. E.—The first work of the geologist in connection with engineering work, that the speaker recalls, was many years ago on the investigation of ground-water supply in Eastern Massachusetts. The case was that of a town which had taken water from the ground near a river and was sued by a riparian owner for damages for diversion of the water. In that case, the town authorities succeeded in proving, with the aid of a geologist, that the stream came underground from the White Mountains, and escaped paying any damage for the water diverted.

The work with which the speaker has had to do recently has been chiefly in connection with studies for reservoirs in the valleys of some of smaller New England streams, tributaries of the Connecticut River. Here, one finds the usual conditions of streams in New England and the eastern part of the United States generally, deep gorges filled with sand and gravel and with rock on the sides. The determination of the location of the bottoms of these valleys is difficult and, in preliminary investigations, it is often important that the work shall be done without too great expense. In one of these valleys, it was found that the bottom was covered with boulders, some of which were very large, beneath 800 ft. of sand and gravel, but, fortunately, in that part of New England, there is no limestone, and none of the soft sandstone has been encountered in the localities examined. The aid of the geologist was very valuable in these investigations.

William W. Brush,† M. Am. Soc. C. E.—This paper is so complete that it does not leave much to be said in discussion, unless one goes outside the subject presented.

The speaker had the good fortune to be in charge of the studies for the City Aqueduct of the Catskill System. Extensive boring investigations were made so that these studies might be as complete as possible. In New York City, Dr. Berkey gave valuable assistance in determining the alignment of the pressure tunnel of the City Aqueduct. Owing to the location of this tunnel and the character of construction adopted, the traffic and life of the city were virtually unaffected during its construction. This is in striking contrast to the conditions in the streets when the many large pipe lines were laid for delivering the water to the smaller distributing mains.

Great difficulty was experienced in obtaining precise information regarding the rock formation underlying New York City from borings which had been

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previously taken, as well as from other excavations. The borings made by the Board of Water Supply have been so recorded and the rock samples so marked that no difficulty should be experienced by those who, in the future, may wish to use them.

Dr. Berkey developed a map of the geology of Greater New York from information that he had obtained before the borings of the Board of Water Supply were commenced. The location of the various rock formations shown on this map differed materially from those that had been given on other maps. The borings subsequently made under Dr. Berkey's supervision for the Board of Water Supply, proved the accuracy of his deductions, although, in many instances, he had meager data from which to draw such conclusions. As a result of the work of the geologist, virtually no unnecessary borings were made.

The lesson to be drawn from the work on the City Aqueduct, as well as that on other parts of the Catskill System, is that the co-ordination of geological and engineering investigations can be used to great advantage and that such co-ordination should be utilized more frequently than has been the practice.

An experience in connection with the water supply system of Brooklyn is pertinent. The insistence on the part of certain persons that the water supply under Long Island comes from some drainage basin on the mainland and passes under Long Island Sound, caused the authorities of New York City to expend a large sum on developing a water supply based on that theory. Two pumping stations were built under a contract calling for the payment for water delivered, and both these stations have been abandoned. If the advice of a competent geologist had been sought, he would have supported the contention of the engineers, that on account of the geological formation an extensive water supply could not be transported from the mainland to Long Island.

The result of the combined study of the geologist and the engineer in the location and design of the City Aqueduct has been satisfactory as to operation. The speaker has been connected with the operation of the Catskill System since it was placed in service in 1917. The pressure tunnel which extends under the Bronx and Manhattan, to the two terminal shafts in Brooklyn, has met all expectations in delivering a large volume of water without serious leakage. Two or three small leaks have developed in the basement of buildings, but the flow from these leaks has decreased each year, and it has not been serious at any time.

THOMAS H. WIGGIN,* M. AM. SOC. C. E.—The Engineering Profession is indebted to the authors for the record of the geological investigations made for the Catskill works. They have properly emphasized the perfect co-operation which existed between the engineers and the geologists and which makes this work a good example of the proper method of conducting such investigations. Recognition is also due to the broad and economic policy of the former Chief Engineer, J. Waldo Smith, M. Am. Soc. C. E., supported by

* Cons. Engr., New York City.

the Commissioners of the Board of Water Supply and by the engineering advisory staff, which made such a thorough investigation possible. Mining engineers have long made good use of geological investigations and core borings. Engineers in other branches have also made increasing use of these aids, but have generally confined this practice to special problems.

One of the earliest examples of a thorough geological investigation was that made for the Wachusett Dam, in Massachusetts, by Professor W. O. Crosby, for the late Frederic P. Stearns, Past-President, Am. Soc. C. E., then Chief Engineer of the Massachusetts Metropolitan Water-Works and afterward Consulting Engineer on the Catskill works. This investigation was made a few years prior to 1900. On the same works, there was a tunnel the investigations for which were less complete. The portal was in rock and it was assumed that, farther on, the rock was higher and that, therefore, the tunnel would be in rock throughout. As a matter of fact, the rock was lower beyond the portal and for about 1 000 ft. remained at approximately mid-height of the tunnel, thus greatly increasing the cost and making an allowance to the contractor necessary in order to enable him to finish the work.

The so-called New Croton Aqueduct, beneath the Harlem River, penetrated rotten rock in a narrow crush zone and had to be placed at a lower elevation. As regards the number and the location of the borings, the rock had been well explored. The importance of the narrow crush zone, however, as it was apparently disclosed by the boring which traversed it, was underestimated as could easily be done under the circumstances. The disaster that occurred in 1908 when the Loetschberg Tunnel, in Switzerland, was under construction, is still comparatively fresh in the minds of engineers. Owing to reliance on prognostications of geologists, unconfirmed by borings, a zone of disintegrated, wet, and flowing rock débris, thought by the geologists to terminate well above the tunnel, was unexpectedly penetrated, and the tunnel was filled so quickly that 25 men were killed.

The Catskill works had all these examples and others from which to profit and they were heeded. It was realized that a new element was injected into the investigations by the unprecedently high unbalanced pressure of 200 to 400 ft. that would exist in the pressure tunnels when they were in service. The speaker had charge of the design of these tunnels and it was his privilege, in the early stages of the geological investigations, to aid the geologists in visualizing these unusual conditions which differed so greatly from those in the ordinary mining or academic problems.

The Rondout Pressure Tunnel presented one of the most complicated geological problems. Preliminary estimates indicated that the cost of the pressure tunnel would be about the same as that of steel pipes. As it developed later, however, steel pipes would have been somewhat cheaper and, if the Catskill Aqueduct type of concrete-covered, mortar-lined, steel pipe, later suggested by the speaker and used for siphons where the geology was unfavorable for tunnels, had then been evolved, it is possible that such pipes would have been chosen instead of the tunnel, as the tunnel was expected to be wet in the section penetrating the Binnewater sandstone and High Falls shale. There is no reason to regret, however, that the more con-

veniently operated and probably more permanent tunnel was built. The geologists and the engineers were brought into definite contact first in studies of the Rondout Pressure Tunnel when the attempt was made to translate the generally qualitative conceptions of the geologist into the quantitative results necessary to estimate the quantity of water to be pumped during construction, lengths of tunnel that would require timbering, possibilities of outward leakage, etc. The stratification of the rock shown on the tunnel profile, which formed the basis of the studies for depth of tunnel, was mainly the result of close co-operation between the Field Division Engineer, Lazarus White, M. Am. Soc. C. E., his Principal Assistant, J. P. Hogan, M. Am. Soc. C. E., and Professor Crosby, Consulting Geologist, in placing and interpreting borings. Professor Crosby and the speaker subsequently discussed the factors affecting the proper depth of tunnel. Professor Crosby's long experience in mines gave him a definite conception as to the probable leakage, and as the result of this conference, a part of the tunnel was placed 100 ft. deeper than had been previously contemplated. The rate of leakage into this tunnel on the incline where the porous Binnewater sandstone and High Falls shale were penetrated, was somewhat underestimated, but the difficulty in removing the water and of obtaining a tight tunnel was perhaps over-estimated. After the experience with wet ground in the Rondout Tunnel, the pressure tunnels to be built subsequently were viewed with less apprehension both by the engineers and the bidding contractors.

Certain pure limestone strata traversed by the Rondout Siphon caused anxiety to the designers, on account of the experience in the Thirlmere Aqueduct, in England, where the concrete made of similar limestone dissolved badly. As the authors state, this limestone was not used in the concrete lining. The rock through which the tunnel passes is still subject to solution, but reliance was placed in a good lining and careful grouting to reduce the circulation to a negligible quantity. Furthermore, the small carbon dioxide content in the water and its reduction by aeration at Ashokan Reservoir will minimize the power of the water to dissolve the limestone. The tunnel is becoming tighter all the time.

The Wallkill Siphon offered no geological difficulties. It was entirely in shale and rightly expected to be largely free from inward leakage. After the rock profile had been determined by a sufficient number of borings, the only important problem was the necessary depth of tunnel below the rock surface. This was a pioneer line of thought with respect to such large unbalanced pressures.

One criterion applied by the designing engineers was that the depth should be such that the weight of earth and rock above the tunnel would offset the upward unbalanced water pressure, estimated as the height of a column from the hydraulic gradient to the level of ground-water. This condition, however, might be met by a deep cover of earth and a shallow cover of rock. Stability and water-tightness demanded a substantial thickness of rock; even a thick rock cover had its possible weakness, as a laminated rock

could conceivably fail by the accumulation of full pressure in a horizontal seam near the top. A failure of this kind occurred later at the shaft of the City Tunnel in Bryant Park, New York City. The minimum depth of pressure tunnel below the top of sound rock was decided as 150 ft. This was an arbitrary thickness suggested by the speaker and accepted by the Chief Engineer and his advisory staff, and, thereafter, became a standard for fixing the depth of other pressure tunnels on the Catskill System, including the City Tunnel.

Moodna Siphon was partly in shale generally like that penetrated by the Wallkill Siphon and partly in granodiorite like that through which the Hudson Siphon was constructed. As described by the authors, the stretch in shale was suspected of harboring a buried gorge of an ancient stream. Many borings were made in the search for that gorge, some of which were only about 150 ft. apart. Luckily no gorge existed, although the reason that the ancient stream left such a broad flat floor in such easy shale digging is still somewhat puzzling.

Borings in the river to discover the Hudson Gorge were unsuccessful, although one hole, beginning with an 18-in. and ending with a 2-in. casing, penetrated more than 760 ft. It took more than a year to put this boring down, and it petered out as boulder after boulder required successive reductions in the size of the casing. At about that time, the speaker was sent West to investigate inclined shafts in the Michigan copper country. This investigation was apropos of the proposal to sink an incline from each side of the Hudson River toward the center, maintaining each incline at a safe distance below the rock surface by making pilot borings from the incline ahead to the rock surface. The speaker gained considerable information about long inclined borings, and it was decided, instead, to make such borings from the vertical shafts already started. Fig. 12 shows how this was done, although it does not explain why two sets of borings were needed. The records of long inclined borings for Michigan mines showed that they curved upward considerably, as shown by Fig. 19. It had been found that close clearances in the first two boring rods, that is, a small difference between the size of hole as determined by the setting of the diamonds and the diameter of rods, high rotary speeds, and light rod pressure resulted in less curvature, but did not eliminate it. In the specifications for the long inclined Hudson borings, a generous size of hole and all the previously mentioned aids to straightness were used. The lower holes shown in Fig. 12 were drilled first. They were expected to curve upward so as to occupy, at the center of the river, a position not much deeper than that explored by the second or upper set of holes. They remained practically straight, however, and went too low, and the upper pair of holes, therefore, was bored to explore the rock higher up and to allow the tunnel to be located at a higher elevation, which was done.

Another consideration in connection with the investigation of the Hudson Siphon, which, in a lesser measure, is pertinent to the other siphons, is the possibility of a buried falls just below the line of borings. It is conceivable that a thin wall of rock might be left on the down-stream side of

the tunnel at such a falls, even if the vertical rock cover was sufficient. The speaker does not recall that such a falls was then regarded as a geological impossibility, although he notes that evidence now available relating to the Hudson Gorge southerly, is regarded by the authors as proving an overdeepening effect rather than a falls at Storm King. The falls was always regarded as a remote possibility and did not affect the profile.

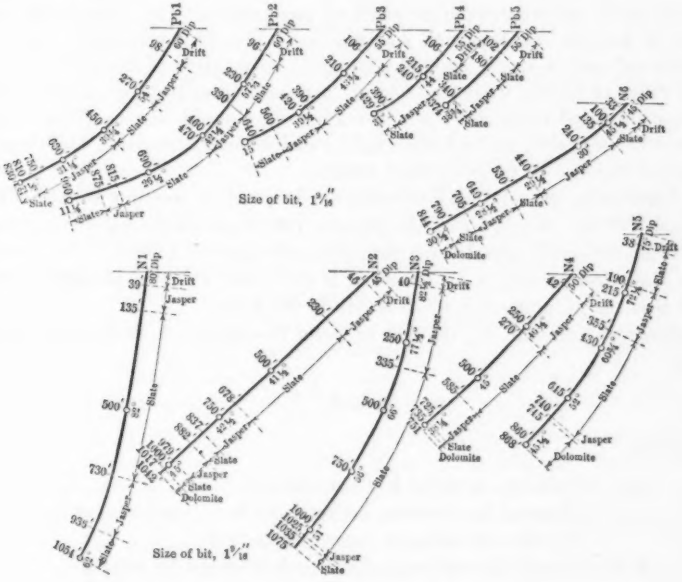


Fig 19.

In describing repairs to the Eastview Tunnel, the authors omitted to state that a water-proofing diaphragm, consisting of three thicknesses of cotton duck impregnated and coated with asphalt in the usual way for such waterproofing, was placed between the original concrete lining and the new brick lining. It was hoped thus to exclude the acid water from the new lining.

JOEL D. JUSTIN,* M. AM. SOC. C. E. (by letter).—The writer was particularly interested in the authors' description of the geologic conditions at the sites of the Gilboa and Devasego Falls Dams. It would be most interesting to learn more of the considerations which led to the abandonment of the Devasego Falls site, where topographical conditions indicated much cheaper structures than those required at the adopted Gilboa site.

The authors state that the ridge of glacial drift which forms in part a natural dam across the valley of the Schoharie at this place, contains a pervious layer about 1 000 ft. wide, 1 200 ft. long, and 30 ft. deep. The top of this pervious stratum is stated to be at about Elevation 1 100.

* With Day & Zimmerman, Holtwood, Pa.

Through the work of Allen Hazen, M. Am. Soc. C. E., Professor Slichter, and other experimenters, the laws of flowing water through pervious soils, sands, and gravels are quite well known. With a thorough sub-surface investigation, such as that at the Devasego Falls site, it is possible to predict, within reasonable limits, the leakage through such a stratum. It would also be possible to determine whether the velocity of any water that might issue to the surface at the down-stream end of the pervious stratum would be sufficient to move any of the constituent particles of the stratum. The value of the water lost by leakage through such stratum could also be determined. With the results of such a study available, it would be possible to determine whether this ridge of glacial drift could safely and economically be utilized as a part of the proposed earth dam, whether a cut-off through the pervious stratum would be desirable, and whether safety and economy required the abandonment of the site in favor of some other.

Apparently, the total head of water on the pervious stratum would not have exceeded 60 ft. Assume that the stratum extends from the up-stream face of the ridge through to the down-stream side, a distance of 1 200 ft. Also, assume that the effective size of the material is 0.10 mm., that the porosity is 32%, and that the ground-water temperature is 60° Fahr.

Slichter's formula* for the flow of water through sands and soils is, as follows:

$$q = K\frac{p\ s}{h}$$

in which,

q = discharge, in cubic feet per minute;
p = difference in elevation between the two points considered or difference in pressure measured in feet;
h = distance through soil along path followed by water;
s = area of saturated cross-section of soil considered; and
K = the transmission constant.

The factor, K, is always the same for the same soil at the same temperature. The transmission constant, as defined by Slichter, is "the quantity of water that is transmitted in unit time through a cylinder of the soil of unit length and unit cross-section under unit difference in head."

The proper transmission constant for the given effective size and porosity, originally determined experimentally, is 0.00405, as obtained from Table 1 of *Water Supply Paper No. 67.*

By making the proper substitutions in Slichter's formula,

$$q = \frac{0.00405 \times 60 \times 1\,000 \times 30}{1\,200}$$

$$= 6 \text{ cu. ft. per min.}$$

If instead of the effective size being assumed as 0.10 mm., it had been assumed as 0.35 mm. (about the size of filter sand), computations would have

* *Water Supply Paper No. 67,* U. S. Geological Survey.

indicated a leakage of a little more than 1 cu. ft. per sec. under the assumed conditions.

The velocity of the water flowing through the soil would be obtained by dividing q by the area of the water passages, or,

$$V = \frac{q}{s\,P}$$

in which,

V = velocity, in feet per minute;
q = discharge, in cubic feet per minute;
s = area of cross-section of soil through which water is flowing; and
P = percentage of porosity expressed as a decimal.

Thus, with the computed discharge of 6 cu. ft. per min., $V = 0.00063$ ft. per min., or less than 1.0 ft. per day, which is less than that required to move any particles of the assumed material. These computations are merely illustrative. No doubt a study along the lines herein discussed was made by the engineers of the Board of Water Supply. It is to be hoped that the details of the investigation and study of the Devasego Falls site will be described, as they would be of great value to engineers having similar problems.

In some sections of the United States, impervious foundations for earth dams are not available and some earth dams must be built of quite pervious material. Where such conditions have been found and properly interpreted by thorough sub-surface investigations, safe dams have been designed and built.

JAMES F. KEMP,* ESQ. (by letter).—This valuable paper summarizes the unusual experience gained in the application of geological investigations to a great engineering problem. The reading of it recalls many lessons which were driven home to those who shared in the early work; and, at least, one geological principle which to that time had not been appreciated at its true value in the United States. Geologists no less than engineers have profited by the work of the Catskill Aqueduct. In discussion, the writer would like to direct attention to one or two of these especial problems.

The selection of the most advantageous place at which to cross the Hudson River was one of the early and, also, one of the most important problems. As noted in the paper, a number of tentative explorations were made above and below the Storm King crossing which was finally chosen. The explorations were made by wash borings, and profiles of the bed-rock bottom based on the results were prepared. The conclusion of those in charge, however, was that by this method the drill might easily come to rest on a large boulder and furnish unreliable evidence regarding the bed-rock unless the borings were checked by the diamond drill or the percussion drill to a depth sufficient, from the thickness and the nature of the rock penetrated, to preclude a boulder. It was learned that, for deep land borings or for borings under a river or other body of water, unchecked wash borings are of little value, especially in glaciated regions.

The geologists also had in mind the importance of finding a crossing for the pressure tunnel where there would be no faults. Breaks in the strata meant

* Prof. of Geology, Columbia Univ., New York City.

places of weakness which, under earthquake stress, might also be places of movement. Unless the selected location should prove practically impossible because of other considerations, such as disadvantageous approach, they desired to decide on a crossing without these structural lines of disturbance. Investigations led to the realization that there were many faults in the part of the Hudson Valley under exploration. Contrary, moreover, to the general belief regarding faults, those discovered were usually not of the normal, but of the reverse, type, and indicated that frequent over-thrusts had been produced in the past. The later and more detailed studies of Dr. Berkey in the West Point Quadrangle and field trips with students in other areas of this region, have abundantly corroborated the early experience on the Aqueduct.

Geological considerations favored a crossing where the same kind of rock was exposed on both banks and presumably continuous beneath the river. The Storm King crossing with its granite on both banks received early preference; but, in so narrow a gorge, one could not ignore the possibility or even the probability of a fault as the line of weakness which originally located the drainage. It was fortunate, however, that subsequent exploration, disclosed no fault beneath the middle and west side and only a small fault near the east bank, which fault fortunately passed upward into Breakneck Mountain and not into the river gravels and silts. Elsewhere, along the course of the Aqueduct many faults and weak crushed zones were found, and from the trouble caused by them, as described in the paper, they abundantly justified the original apprehensions.

The wash borings in the tentatively explored crossings above the Highlands disclosed no reason to expect unusual depth to bed-rock in the narrow Storm King Pass. Farther up stream the valuable record of the exploration to bed-rock at the Poughkeepsie Bridge, which were contributed by John F. O'Rourke, M. Am. Soc. C. E., in 1888,* also indicated the bed-rock from shore to shore, at about 140 ft. below high-water mark. The natural inference was that the granite at Storm King and Breakneck Mountain, which extended across the 2 000 ft. of river and which was the hardest rock in the stream from the Adirondacks to Cornwall, would furnish a gigantic riffle rather than an unusually deep depression. In the earliest work, this consideration gave added support to the choice of the Storm King site for the pressure tunnel. The reasoning that would have been followed at that time by practically every American geologist was that the bed-rock channel had been eroded by running water; that there must be a down grade to the sea; and that the land must have stood higher than it does at present with reference to sea level, so that the rivers could drain off. A progressively deeper channel was expected through the lower reaches of the river past New York City and out beneath the present ocean bottom. Geologists knew of the off-shore gorge of the Hudson, in the Continental Shelf, as it had been described years before,† but they were not

* "The Construction of the Poughkeepsie Bridge", *Transactions*, Am. Soc. C. E., Vol. XVIII (1888), pp. 199–215, especially p. 202.

† A. Lindenkohl, "Geology of the Sea-Bottom in the Approaches to New York Bay", *American Journal of Science*, Vol. 29, p. 475 (1885), and "Notes on the Submarine Channel of the Hudson River, etc.", *loc. cit.*, Vol. 41, p. 489 (1889); and J. W. Spencer, "Submarine Great Canyon of the Hudson River," *loc. cit.*, January, 1905, pp. 1–15.

prepared for the great depth, 800 ft. ±, which the drill subsequently revealed. This great depth led them to two possible conclusions, namely, the existence of a deep narrow gorge, deepening constantly to the south, past New York City to the sea, or, an over-deepening of the gorge at Storm King by sub-glacial erosion and the gradual rise of the bed-rock to and beyond Manhattan Island, as the southward moving glacier grew thinner and thinner and less effective as it approached the Terminal Moraine. The writer is of the opinion that the latter explanation is correct. Above the Storm King crossing is the broad open bay of Newburgh, once undoubtedly filled with southwardly moving ice. The abrupt front of the Highlands crowded all the ice below the tops of the mountains into the narrow Storm King Gorge and created conditions exceptionally favorable for sub-glacial scouring and erosion. Access has since been gained to the records of the borings made by the Pennsylvania Railroad Company opposite 33d Street, Manhattan, for its tubes, but there is a gap of 1200 ft. in mid-stream in which it cannot positively be stated that a deep narrow gorge may not exist, although that seems to be unlikely.* The bed-rock on the west side of the interval was 301 ft. below the surface, and that on its east side 287 ft. Nowhere else was the bed-rock so deep. Geologists still hope that some time a diamond drill boring may penetrate and demonstrate the bed-rock in mid-stream. At present, however, it may be stated that the Storm King crossing and its great depth to bed-rock beneath the river has convinced the writer and many other American geologists of the soundness of the belief in the sub-glacial erosion by ice.

The borings in minor stream bottoms in the approaches to the Hudson River and the departures from it, and for the tentative sites for the great Ashokan Dam invariably revealed buried bed-rock channels, sometimes coincident with the general location of the present stream, and sometimes at distances from it. Geologists have thus been able to reconstruct an interesting system of pre-glacial drainage active when the land stood higher than it does to-day with reference to the sea level.†

The geological sections in the Hudson Valley, in the Wallkill Valley, and in the Upper Esopus Valley, are comparatively simple; but when the Rondout Valley is crossed, the section both in number of formations and in faulting becomes exceedingly complex. It was explored, therefore, with great thoroughness by the drill, and the results were accurately drawn in profile before bids for the sub-divisions of the pressure tunnel were invited. Contractors were thus enabled to bid with full knowledge of what they would find regarding the depth and character of rock. The geologists realized that the Binnewater sandstone was practically certain to be a wet formation. The precaution was

* In 1910, these data were placed in the hands of the late G. Sherburne Rogers, at the time one of the graduate students of Columbia University, and was the basis of the paper by him, entitled "The Character of the Hudson Gorge at New York City," which was published in the *School of Mines Quarterly*, November, 1910, pp. 26–42.

† "Buried Channels Beneath the Hudson and Its Tributaries," by the writer, *American Journal of Science*, October, 1908, pp. 301–323; "Geology of the New York City Aqueduct," by Charles P. Berkey, *Bulletin 146*, New York State Museum, 1911, which gives numerous local details. For a review of buried channels elsewhere in the region, see "Buried Channels in the Northeastern States," by the writer, *Proceedings*, Wyoming Historical and Geological Soc. of Wilkes-Barre, Pa., Vol. 14, pp. 1–20 (1913). These phenomena are of much interest to civil engineers in connection with dam sites, and many problems of water supply.

thus adopted by the engineers of dipping down with the tunnel as they neared this sandstone and, after installing pumps, to tap the sandstone on the up grade. The water which rushed in, was thus promptly cared for, and no delay in the work resulted.

Many other individual problems are admirably set forth by the authors, to whom engineers and geologists everywhere are under a great debt. The problems selected for discussion by the writer are those, however, which occur to him as of especial interest and importance to both groups.

THADDEUS MERRIMAN,* M. AM. SOC. C. E. (by letter).—Attention has been called by Mr. Justin to the general question of underground flow, and he has expressed a wish to know what theoretical investigations, if any, were made in order to determine the feasibility of the Devasego Dam site. The controlling reasons for the adoption of the site at Gilboa in preference to that at Devasego were:

First.—Sub-surface investigations at Gilboa showed this site to be much the better. No uncertainties such as those disclosed at Devasego were revealed by the borings.

Second.—The Gilboa site would furnish a larger reservoir with more storage and, at the same time, increase the available drainage area from 243 to 314 sq. miles.

The situation at the Devasego site was as follows: As stated by the authors, the preliminary borings had disclosed the existence of a large body of doubtful material about 30 ft. thick and 1 200 ft. wide. In order to determine the perviousness of this ground, a number of tests were made by pumping water into the drill casings. By noting the pressure at the top of the casing, the quantity of water pumped, and the rise of the ground-water table at other holes in the vicinity, it was possible to reach reasonable conclusions as to the continuity of the porous zone and its water-carrying capacity.

Twelve of these tests were made after the drill casings had been lifted and the drill holes in the rock had been completely filled with clay in order to prevent the escape of water through the rock joints. The results of these tests are summarized in Table 1.

TABLE 1.

Test No.	Length of test, in hours.	Rate of pumping, in gallons per minute.	Pressure at top of drill hole, in pounds per square inch.	Number of responses in other holes.
1	63	72	Negative	2
2	59	37	15	2
3	48	75	Negative	3
4	55	90	"	8
5	56	91	"	10
6	24	83	25	1
7	51	89	Negative	11
8	28	87	"	12
9	23	86	"	8
10	22	82	12	5
11	31	64	30	3
12	31	137	Negative	5

* Chf. Engr., Board of Water Supply, City of New York.

The tests, summarized in Table 1, showed that the doubtful material was exceedingly porous and could not be depended on alone to serve as part of any dam. For instance, Test No. 7 showed that 89 gal. per min., pumped for 51 hours—a total of 272 000 gal.—raised the elevation of the water-table in eleven other holes an average of 0.46 ft. over an area of about 15 acres. The most distant hole in which a rise was noted was 900 ft. from that into which the water was pumped. Although these tests did not quantitatively disclose the capacity of this material for carrying water, they did show that at many points, with little or no resistance, it would take from 80 to 130 gal. of water per min. through a 2½-in. pipe, and that this water would flow freely for long distances and over large areas.

Although the methods suggested by Mr. Justin may be applicable to cases in which sand and gravel have been uniformly laid down, they cannot be adapted to locations such as that at Devasego, or, in fact, to any location in a glacial deposit, unless the facts as to the uniformity of that deposit are known.

The problem at Devasego was generally similar to that described by the authors at "The Tongore Alternative Site" of the Ashokan Reservoir. The preliminary borings at this alternative site disclosed conditions which were interpreted as indicating a doubtful and undesirable situation. The site, however, offered such other advantages that further examination seemed to be desirable. A shaft, therefore, was sunk to the material which had been shown by the borings to be pervious. In commenting on the conditions at the time (1906), Dr. Berkey wrote as follows:

"The conditions reported in the new large shaft (Tongore) in the gravel beds now being penetrated, *i. e.*, at first 'no water', and then a little deeper an 'abundant flow,' I interpret as follows: It is not an isolated pocket or lens, else it would have carried water full from the first. It must be a fairly continuous porous zone with large feeding connections else it would run dry, and with easy discharge, else it would have risen above the first gravels. Therefore, it must be a rather well marked subterranean water passage or perhaps a porous zone of considerable extent. Such conditions it seems to me make an impervious core wall to bed-rock a necessity, and its construction a matter of considerable difficulty. In this site, because of the small cross-section of the ridge, there is little chance for the interlocking of layers or the blocking of the porous ones by a till barrier."

These two cases serve to illustrate the necessity of supplementing the results obtained by the ordinary wash-borings whenever the conditions are doubtful. They also serve to show that theoretical considerations alone are not to be depended on as trustworthy guides.

CHARLES P. BERKEY,* ESQ., and JAMES F. SANBORN,† M. AM. SOC. C. E. (by letter).—The writers wish to express their appreciation of the comments that have been made by the engineers who have contributed to the discussion. Altogether important contributions have been made, and the writers willingly accept the corrections, criticisms, and additions.

In the beginning, it would have been possible to increase greatly the scope and detail of the paper. Many items of more than local or personal interest were deliberately omitted in order to keep the paper within moderate length

* Prof. of Geology, Columbia Univ., New York City.
† Cons. Engr., New York City.

and also to reserve the available space for more careful attention to certain items that seem to illustrate better its purpose.

The writers have held strictly to the plan of laying a rather simple foundation picture of the nature of the enterprise, and the conditions under which it had to be developed. Following that, the major instructive geologic incidents and problems that seemed to promise a contribution of possible value to the Engineering Profession were treated. It was believed that examples of the interrelation and interdependence of geologic and engineering factors were exhibited so well in certain parts of this project that they might serve a good purpose in illustrating appropriate methods, splendidly effective co-operation, and successful solutions of many obscure and somewhat baffling natural conditions.

It is the writers' belief, based on a great many years of observation, that neglect of geologic factors or inadequate study of them is a common fault in a majority of engineering undertakings. Many serious blunders may be traced to this as a fundamental cause, and it is hoped that the illustrations given in connection with an important project, such as the Catskill Water Supply, which is now known to have been very successfully conducted, may be serviceable. There is no desire to over-emphasize the contribution of geology to that success; that it was a contribution of no little importance is so generally attested by the opinions of the engineers in charge and by the results secured, that the writers feel fully justified in stating the facts and in separating the principal geologic problems as found in this work from the obscuring mass of other problems and data, so that one may the more clearly see their bearing.

The writers do not assume to claim that a geological study is the cure for all difficulties. Neither do they assume to suggest that geologic opinion is to take the place of properly organized exploration. This is a mistake commonly made. It is just as extreme and unreasonable as the attempt to ignore the geologic factors. It is not unusual to hear engineers say that even if a geologist is employed explorations must be made, as if that was some sort of legitimate reflection on the geologist and sufficient excuse for the engineer. Yet, that is exactly what should be done in the average engineering case. If he is competent, the geologist can indicate not only what kinds of explorations should be made, but also where they must be made to accomplish the most useful returns in the most economical way. He should be able, also, if he is familiar with the needs of the engineer, to interpret the data observable in the field and obtained by exploration, more logically and more reliably than one who is not thus trained. The geologist, therefore, should not be regarded as a substitute for exploration, or an excuse for loose methods, but an interpreter of geologic conditions, an aid in successful investigation, and a critical adviser in the matter of design, methods, and contracts that have to do with construction in the ground.

Many engineers of long experience become familiar with physical conditions and the laws of ground behavior. They are normally close observers and rigid reasoners and are exceedingly cautious in drawing conclusions. They are preeminently qualified in these respects for just such kind of work as falls also to the geologist; and it often happens that little more than support can be

given to the conclusions already reached by such engineers. It so happens, however, that it is not usually possible for a man in one great field of endeavor to become a master of other, even closely related, lines. He cannot feel certain that all its possibilities are met and that his conclusions are reliable. To such a man, a competent geologist can render real service. It sometimes happens that an engineer is not well enough informed or experienced to appreciate what the problem is, or even that there is a problem at all or that it can be solved, and, for this reason, among others, additional opinion is not asked. Sometimes, one has to ask permission to help such men, or one is asked to explain the cause of the trouble after it is too late for advice to serve any purpose.

The ideal situation is one in which the engineer, already comparatively well grounded in all the major features of the problem to be encountered, is still able to appreciate that a fuller or more critical study might materially improve the chances of success, or allow the design to be modified better to suit the conditions, make the contracts more definite, secure more economical treatment, or conduct the work on safer lines. Wherever this broad view is taken, and adequate consultation privileges are conferred on a competent, experienced geologist who appreciates the point of view and purpose of the engineer and will confine himself to the practical questions of the case, material service can be rendered in phases of the undertaking that cannot be reached any other way.

In an important case, where all the facts and conditions are not perfectly clear, the geologist should know enough not to give a final opinion without additional exploratory evidence; and the engineer should be wise enough not to treat an opinion as if it was a proven fact and then blame the geologist for mistakes which belong to both. The average responsible engineer has been found by the writers to be an unusually intelligent man who readily appreciates the distinction between facts of observation and inferences, and between conclusions based on close, logical reasoning and hypotheses based on more or less incomplete or inadequate data. All these things have to be dealt with, however, and it is peculiarly the province of geology to deal with many of the less conclusive hypotheses, not at all because the geologist wishes it to be so, or because it is his habit of mind, but simply because many underground conditions are obscure and because only a few of the many needed facts in a given case can be found by any method at any one's command. Thus, the geologist does the best he can with what he can get, in the belief that this, also, will be helpful.

The paper has been written largely to explain and illustrate that part of the work on the Catskill Aqueduct which normally came under the eye of the advisory engineering geologist. There is like work to be done by some one in every project of a similar nature. The writers are certain that a greater amount of such help should be obtained in most projects having to do with the ground, and they hope that the illustrations given by them, together with the comment made by those who have so liberally and instructively discussed the paper, may help enforce this observation and thus make the contribution something of educational value to younger men who, in the future, are to be responsible for large public undertakings.

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Paper No. 1510

SOME NOTES ON THE LOCATION AND CONSTRUCTION OF LOCKS AND MOVABLE DAMS ON THE OHIO RIVER, WITH PARTICULAR REFERENCE TO OHIO RIVER DAM NO. 18.

By William M. Hall,* M. Am. Soc. C. E.

With Discussion by Messrs. Thomas P. Roberts, Earl I. Brown, William W. Harts, L. M. Adams, C. I. Grimm, B. F. Thomas, Malcolm Elliott, Frederick B. Duis, W. H. McAlpine, Gardner S. Williams, Morris Knowles, and William M. Hall.

Synopsis

This paper presents briefly a history of the improvement of navigation on the Ohio River.

To maintain a navigable depth during the low stages of the river and to retard as little as possible open-river navigation during the high stages, movable dams with locks were constructed.

The considerations influencing the location, design, and construction of these dams, together with some notes on operation, are discussed by the writer, with particular reference to Ohio River Dam No. 18.

Introduction

Historical.—The first work of aiding and improving navigation on the Ohio River consisted in the removal of snags, boulders, and bars, the building of dams for closing the smaller and less important channels at islands, the building of spur-dikes to narrow the channel at some of the bars and shoals, and, in later years, the dredging of shoals and the building of ice piers to protect boats from floating ice when in "harbors" or protected places alongshore. All these methods failed to maintain a minimum depth of 3 ft.

* Parkersburg, W. Va.

for navigation during dry seasons and, as a last resort, the more expensive method of improvement with locks and dams was undertaken.

The first appropriation by the National Government for the Ohio River was made in 1824, and the improvement of the river by the open-channel method was commenced in 1827. From 1825 to 1830, a private company, under a charter from the State of Kentucky, built the Louisville and Portland Lock Canal for carrying traffic around the Falls of the Ohio (Mile 604 below the head). On February 8th, 1885, the United States acquired the stock of the Company and thereafter operated and improved the canal.

In 1870, the late W. Milnor Roberts, Past-President, Am. Soc. C. E., made the first written recommendation of record for changing the method of navigation to that of locks and movable dams. In 1874, the late Maj. W. E. Merrill, Corps of Engineers, U. S. A., M. Am. Soc. C. E., recommended the improvement of the upper river from Pittsburgh, Pa., to Wheeling, W. Va., by a system of thirteen locks and movable dams. The River and Harbor Act of March 3d, 1875, appropriated $100 000 for starting construction on Lock and Dam No. 1 at the head of Davis Island, about five miles below Pittsburgh, which lock and dam was opened for operation on October 7th, 1885.

Since that time, by enactments at irregular intervals, locks and dams to No. 39, inclusive (Mile 529), and Nos. 41, 43, 44, 45, and 48, have been placed under construction. Of these locks and dams, thirty-three have been placed in operation.

Physical Features.—The Ohio River (Fig. 1) is formed by the confluence of the Allegheny and the Monongahela Rivers in Western Pennsylvania. It flows in a general southwesterly direction and empties into the Mississippi River, the two, near their confluence, forming the southern boundary of Illinois. The total length of the Ohio is 968 miles, and it has a drainage area of 203 900 sq. miles. The discharge of the head varies from about 1 600 sec-ft. at low water to 440 000 sec-ft. during flood, and, similarly, at the mouth, from 27 500 to 1 500 000 sec-ft. At low water, the average fall is, as follows:

	Miles.	Inches per mile.
Pittsburgh to Wheeling.....	90	11½
Wheeling to Cincinnati.....	378	5½
Cincinnati to mouth*.....	500	4

The width of the channel increases gradually from about 850 ft. near the head to 5 900 ft. at a point 22 miles above the mouth. The extreme stages of the river as given by gauge records at various points are, as follows:

Pittsburgh, Pa.....	(Mile 1),	highest stage	35.5 ft.	in	1907
Parkersburg, W. Va..	(" 186),	" "	58.9 "	"	1913
Cincinnati, Ohio.....	(" 469),	" "	71.1 "	"	1884
Louisville, Ky., below Falls.....	(" 604),	" "	70.6 "	"	1884
Cairo, Ill.....	(" 968),	" "	54.8 "	"	1913

* Including 26 ft. in 2 miles of the Louisville Falls.

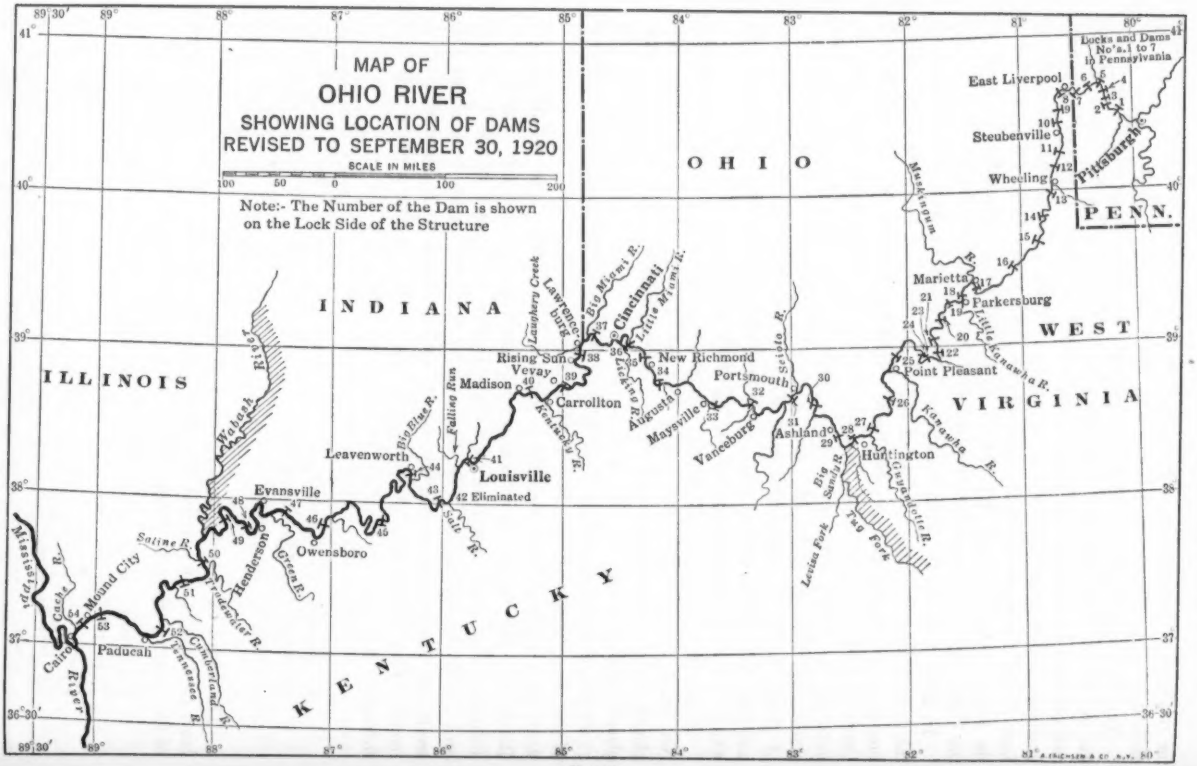

MAP OF
OHIO RIVER
SHOWING LOCATION OF DAMS
REVISED TO SEPTEMBER 30, 1920
SCALE IN MILES
100 50 0 100 200
Note:- The Number of the Dam is shown on the Lock Side of the Structure
Locks and Dams No's.1 to 7 in Pennsylvania
OHIO
INDIANA
ILLINOIS
KENTUCKY
WEST VIRGINIA
PENN.
East Liverpool
Steubenville
Wheeling
Pittsburgh
Marietta
Parkersburg
Point Pleasant
Cincinnati
Lawrenceburg
Rising Sun
Vevay
Madison
Carrollton
New Richmond
Portsmouth
Ashland
Huntington
Maysville
Vanceburg
Louisville
42 Eliminated
Leavenworth
Evansville
Owensboro
Mound City
Cairo
Paducah
Henderson
Saline R.
Green R.
Tradewater R.
Cumberland R.
Tennessee R.
Wabash R.
Big Blue R.
Falling Run
Laughery Creek
Big Miami R.
Little Miami R.
Kentucky R.
Licking R.
Muskingum R.
Scioto R.
Big Sandy R.
Guyandotte R.
Kanawha R.
Levisa Fork
Tug Fork
Little Kanawha R.
Mississippi R.

Fig. 1.

During the low-water season, which occurs between July and December, the stage rarely falls to the zero of the gauges, but in many seasons it has been below 2 ft. for days and occasionally for months.

Table 1 which covers a period of several years before any dams were built, indicates the duration of navigation for 9-ft. stages or higher.

TABLE 1.

	Miles below head.	Days over 9-ft. stage.
Dam No. 1	6	79
Dam No. 6	28	117
Wheeling, W. Va	90	119
Parkersburg, W. Va	183	141
Portsmouth, Ohio	353	216
Cincinnati, Ohio	469	248
Louisville, Ky., Head of Falls	601	97
Evansville, Ind	790	198
Cairo, Ill	968	302

Extent as Water Route.—The inland water route for greatest tonnage and importance in the development of the United States is that from the easterly end of Lake Erie to the westerly end of Lake Superior. Second to it, is the route from Western Pennsylvania down the Ohio and Mississippi Rivers to the Gulf of Mexico. As Professor Hurlbert has well said in his history:* "From whatever standpoint one views the Ohio River, it has a most interesting history; but of them all none is more attractive or important than that from which it appears as a strategic avenue of National expansion."

In the settling and building of the Middle West, the Ohio River was one of the great highways, and when National, rational, and wise traffic regulations have been adopted, it will continue to exert a great influence on traffic and the upbuilding of the country through which it flows. Possibly no subject of National policy requiring large expenditure has been more abused, distorted, and used for political and private ends, during the past fifty years, than the river and harbor appropriations. Notwithstanding that condition, the project as wrought out for improving this great river appears creditable to the country and to the Governmental agents in charge. From the viewpoint of the engineer employed on the improvement now projected and in progress during the past two decades, it is no less interesting than it was to the explorer and pioneer, one to two centuries ago.

The mileage of navigable water in the Ohio River System is about as follows:

Ohio River	968
Allegheny River	38
Monongahela River	130
Muskingum River	90
Little Kanawha River	108
Great Kanawha River	90
Big Sandy River	27
Carried forward	1451

* "The Ohio River."

Brought forward	1 451
Tug Fork	12
Levisa Fork	18
Kentucky River	268
Green River	196
Rough River	29
Barren River	29
Wabash River	100
Cumberland River	518
Tennessee River	650
Total	3 271

The Ohio River and the Great Lakes Systems will probably be connected between Pittsburgh and Lake Erie and, possibly, across Illinois, thereby making an inland system of great National importance. These projects are now being strongly advocated.

Commerce.—The commerce of the Ohio River is divided into two general classes: Package freight, loaded on the decks of steam and gasoline boats, and bulk freight, such as coal, timber, iron, and petroleum products, moved by barges.

The boats that carry package freight and passengers range in size from the diminutive gasoline motor-boat to steamboats 320 ft. in length. The barge freight is carried largely on coal boats about 26 ft. wide by 130 ft. long and from 8 to 12 ft. deep.

The fluctuation in tonnage is shown in the Annual Reports of the Chief of Engineers, U. S. A., and is indicated by the extract given in Table 2.

TABLE 2.

Down Stream:	1917:		1916:		1894:
	Through lock.	Through pass.	Through lock.	Through pass.	Up and down stream.
Tons	1 316 321	1 925 623	1 397 067	2 760 797	7 795 501
Ton miles	115 187 305	192 030 473	40 270 649	506 599 000	.....
Average haul	87.5	99.7	28.83	183.50	.....
Passengers	41 796	47 116	37 253	456 468	1 033 492
Passenger miles	1 329 650	2 107 412	2 373 053	8 291 800	.....
Average haul	31.8	44.7	63.07	18.16	.....
Up Stream:					
Tons	559 713	797 217	1 176 866	710 183	.....
Ton-miles	29 625 309	41 647 722	31 654 970	19 858 843	.....
Average haul	52.9	52.2	26.89	27.96	.....
Passengers	20 702	69 785	39 473	625 322	.....
Passenger miles	1 355 595	4 113 796	2 422 935	10 592 707	.....
Average haul	65.5	58.9	61.38	16.93	.....

Since 1894, the greatest decrease in tonnage has been principally in the coal from Pittsburgh down stream. The freight moved in 1894 was about 3 500 000 tons, of which more than 3 000 000 tons were of coal. Within the past decade the shipment of coal from the Pittsburgh District down stream has been stopped almost entirely by the adoption of a policy to conserve it

for home consumption. This policy cut off at one stroke about one-half the tonnage of the river.

The writer thinks that the building of locks and dams is not rapidly increasing the tonnage, nor will it do so for package freight until the policy in reference to its movement is more satisfactorily administered. Under present methods of operation, no company carrying passengers and freight in less than barge loads, has built up a trade and held it at a fair and profitable rate.

In 1909, the Receiver of the Pittsburgh and Cincinnati Packet Line, the largest line on the upper river, gave the following reasons for this financial condition:

1.—The non-interchange of traffic between river and rail companies;
2.—Large terminal charges by some towns;
3.—Low railroad rates at competitive points; and
4.—Suspension of boat traffic by low water.

Another reason occurs to the writer: Many boat transportation companies have built up a considerable package traffic, with the almost unfailing result of having it destroyed by a second company entering the same trade and taking a large part of the first company's business, by cutting prices to such a rate as to destroy both companies. Until these wrongs are adjusted or remedied in some way, the river will continue to carry a diminutive package commerce and fail properly to serve the public or earn a fair value on the large expenditures for its improvement.

Unfortunately, neither the Engineer Department, nor other Government agency, manages or seriously concerns itself with any part of the transportation problem other than the preparation of a sufficient waterway. This question seems to be left mainly to the boatmen, members of Congress, and the annual two-day meeting of the several "Valley Improvement Associations". It is the writer's opinion that if the package-freight transportation rights of the river with its tributaries were awarded under proper and competitive methods to one reliable company, and operated under Government supervision, providing for efficient and reliable service at fair rates, the tonnage would be greatly increased. Should such a privilege for a long period be made to one of the largest and best railway companies crossing the river, thereby increasing its operating lines more than 3 000 miles, there is little doubt but that a great expansion in the tonnage would result. Such a policy, however, is in direct conflict with the popular policy of a free river and free commerce to every boat, which has been in effect since the formation of the Government; therefore, to effect the suggested change will require a struggle, like every other reform that brings great good. However, recent years of Government regulation of railway rates; of diminishing railway expansion and exploitation; of shrinkage in railway revenues and profits; and the more extensive realization by the public that the people along efficiently and reliably operated transportation lines profit many times more than the owners of the lines, will probably cause acquiescence to such a change much more readily than it would have done a few years ago. Since freight transportation by boat for short runs of less than about 500 miles, is cheaper and quicker than by railroad,

it follows as an incontrovertible conclusion that such transportation should be used and will be used when the right policy is evolved. Some Government agency should certainly be charged with the responsibility.

The capacity of the river for package and barge freight, when the improvement is completed, would easily be 25 000 000 tons annually and possibly double or treble that quantity. However, during the months the dams are up, the lockage capacity would be much below that rate.

Project for Improvement.—The appropriation by the Government for the improvement of the Ohio River for all purposes to June 30th, 1918, totals $59 500 000.

Since the design and construction of all the locks and dams on the upper half of the river, above the sill levels, are nearly the same, it is evident that the large differences in cost, as shown in the Annual Report of the Chief of Engineers for 1920, are largely due to the difference in the foundations. Other causes affecting such differences are prices of work as contracted, materials purchased, and management.

From 1890 to 1908, the policy was to construct first those dams below the large cities and below mouths of large navigable tributaries. This policy was modified later and now has become fixed in favoring a continuous order from the head of the river.

It is well known that a project substituting fixed dams for movable dams would be less expensive; but the requirements of navigation and the popular fear of an increase in flood heights made the building of fixed dams in this river appear at that time to be impracticable.

The project for the improvement as now contemplated was adopted by the Act of June 25th, 1910, per the report of the Lockwood Board.* The total number of locks proposed was 54, but by the Act of August 8th, 1917, it was proposed to construct one lock and fixed dam with high lift to replace Dams Nos. 1 and 2, and by the Act of September 12th, 1913, Dam No. 42 was omitted by increasing the lifts of several others, Nos. 43 to 46, inclusive.† It is proposed to omit Dam No. 40 by the increased height made in Dam No. 41. Before the system is completed, other modifications may make further reduction in the number; the omission of Dams Nos. 51 to 54, inclusive, now appears to be quite probable.

In 1894, there were in operation Lock and Dam No. 1 near Pittsburgh and Dam No. 41 at Louisville, Ky. In 1912, there were 13 locks in operation and in March, 1921, 33 locks had been completed, making a total of 432 miles of improved river.

Principal Features.—The principal features of a complete lock and movable dam with equipment, as constructed on the Ohio River, are the lock, navigable pass, weir, abutment and its bank protection, upper and lower guide-walls of the lock, lock esplanade and esplanade slope, bank protection below and above esplanade, power house, operating machinery, maneuvering boat and appliances, men's quarters and grounds, and fuel for power, heat, and light. Besides these auxiliaries, other important features to be arranged for or to be provided

* House Doc. No. 492, 60th Cong., 1st Sess., and additional information in House Doc No. 1159, 62d Cong., 3d Sess.

† House Doc. No. 1695, 64th Cong., 2d Sess.

are a turbine with complete auxiliary power machinery which replaces steam or natural gas engines when the dam is up, telegraph, telephone, mail service, highway and rail connections, farm lands, schools, churches, and amusements.

The original provisions called for a minimum navigable depth of 6 ft., a minimum width of navigation channel of 600 ft., and lock chambers 110 ft. wide by 600 ft. long between gates. The River and Harbor Act of March 2d, 1907, provided for an increase of navigable depth to 9 ft., and for moving the existing and prospective traffic, lock chambers 110 ft. wide by 600 ft. long between gates and navigable channels from 600 to 700 ft. wide, have become standard.

With the foregoing remarks on transportation conditions and on the plans and projects for improvement, a discussion of the principles influencing the location and design, together with some notes on construction and operation, with reference to Ohio River Dam No. 18, will be given.

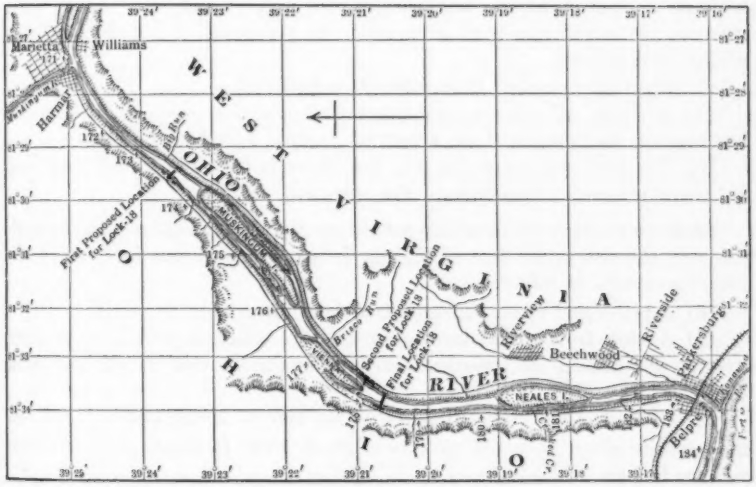


FIG. 2.

OHIO RIVER DAM NO. 18.

Location.—When the construction of Dam No. 18 was assigned to the writer in August, 1901, its proposed location had been changed from Moore Junction to the foot of Vienna Island (Fig. 2). Core drill borings over the Vienna site and the site finally chosen indicated that at the latter location was the highest ledge between the mouth of the Muskingum and the mouth of the Little Kanawha, the extreme limits for the location.

The final location having been decided on, surveys were made at the site chosen for the purpose of preparing estimates and plans. Topography on a 2-ft. contour interval was taken over the site, together with soundings taken

20 ft. apart over the areas covered by the proposed works and 40 ft. apart elsewhere.

Core drill borings were made on the center line of the upper and lower guide-walls, the center lines of the river and land-walls of the lock, the center line of the abutment, one hole at each gate recess, and two lines of holes, one above and the other below the proposed crest of the dam. The drilling was done from an open barge, 14 ft. wide and 23 ft. long by 3 ft. deep, with the floor a few inches from the keel, equipped with a machine drill operated by a gasoline engine.

It will be observed that the lock at Dam No. 18 is located on the concave shore. The principal reasons for placing it on the concave rather than on the convex shore are, as follows:

(1) The depth of water across the river admitted placing the sills at the height desired.

(2) It is on the extreme end of a bar with deep water immediately below, with no danger of having any part of the wickets and sill "sanded up"; whereas, the Island site is on a bar with the possibility of "sanding up" the wickets and pass sill.

(3) A saving in cost of from $200 000 to $400 000.

(4) As built, all the masonry has rock foundation, whereas, if the lock had been on the opposite shore, a part of it would have been on piles.

(5) The lock and approaches are less likely to "sand up", and, therefore, are less expensive in maintenance and operation.

Aside from the more favorable conditions for the foundations, in comparison with the site at the foot of Vienna Island, other reasons in favor of the site chosen are, as follows:

(6) A better and easier approach from both directions.

(7) A view from a distance up stream of 4 miles, and of 2 miles down stream, whereas, on the opposite shore, the up-stream view would have been ½ mile or less.

(8) Ground above high water lies within 200 ft. of the lock, whereas, on the opposite shore, it would have been about 1 000 ft. away, a great disadvantage in time of floods.

(9) A better location for the power house and dwellings which are above high water and near the lock and power house.

(10) It is 5 miles above the business part of Parkersburg, W. Va., and the mouth of the Little Kanawha River, at the upper end of the plateau on which Parkersburg is being built, which makes its location such as to be assured that, in years of future growth, that city's harbor will not be cut in two; and it is 7 miles below Marietta, Ohio, and the mouth of the Muskingum River, which is a favorable distance as to pool height and pool fluctuations for the mouth of the Muskingum and the wharf and harbor of the town. It is also so favorably located in reference to Parkersburg as to have city telephone service, city natural gas for heat, light, and power, and interurban trolley car service.

(11) The topography of the lock grounds is more adaptable to a pleasing appearance.

(12) The railroad spur to the power house is much shorter and less expensive to operate.

(13) It is near enough to the Muskingum River to give sufficient depth on the lower gate-sill of Lock No. 1 of that river, and yet far enough removed to reduce the annoyance and danger from drift, which would have existed had it been within 2 miles of the mouth of that river.

(14) Its distance from the mouths of the Little Kanawha and the Muskingum Rivers renders the annoyance from the maneuvers of the dam less than would occur to either if it was closer.

Design.—A theoretical detailed computation and design* for the following parts of a set of plans of a typical Ohio River lock and dam, as reported in 1912, by Capt., now Maj., L. M. Adams, Corps of Engineers, U. S. A., M. Am. Soc. C. E., have been made: Rolling gates; lock walls; gate track foundations; river wall power house; steam and water power plants; valve operating mechanism; gate operating machinery; analysis of design of wicket, horse, and prop; pile, arched cellular, and solid concrete pass foundation; and 91-ft. bear-trap weir and foundations.

In general, these parts are common to all locks and dams, except those which have mitered gates and those which do not have river wall power houses.

This information is a part of a public document and for that reason will not be repeated in this paper. However, the considerations and methods used in the determination of the dimensions of the various parts of the dam will be taken up in detail.

In regard to the movable parts of the dam, the writer reported to the District Officer, Maj., now Col., G. A. Zinn, Corps of Engineers, U. S. A., in part, as follows:

"In studying this design careful consideration has been given the designs of the Upper Ohio River dams, the Great Kanawha dams, and the Louisa Dam on the Big Sandy. You have already approved of the height of the sill for the pass, and of building the pass 600 ft. long of Chanoine wickets on a concrete foundation. It is therefore necessary now to discuss only the design of the weir and abutment."

In designing the weir, conditions of first importance to be fulfilled, as reported by the writer, were as follows:

1.—To regulate perfectly the height of the water in the upper pool, at all stages of the river at which it may be desirable to keep up the pass. In this case, it does not seem that it will ever be necessary to keep the pass wickets up when the river is above a 10-ft. stage.

2.—To place the weir sill so low that when the pass wickets are down, no boat or fleet will be stopped by the "swell head" or increased velocity of water through the pass.

3.—To make the dimensions of the weir, pass, and lock such that, with the lock-gates open and the dam down, the velocity of the water will not interfere with navigation. It is also desirable to have this condition fulfilled with the dam down and the lock-gates closed.

* House Doc. No. 1159, 62d Cong., 3d Sess.

4.—To place the weir sill sufficiently high to prevent its "sanding up". Ease of operation and regulation make it desirable to place the weir sill as high as possible.

5.—To arrange the weir for passing drift and floating ice with the least danger to the structure and the greatest ease to the operatives.

6.—It is desirable to place the sills of a bear-trap or other gate maneuvered by power, and a part of the weir sill adjacent to the pass, so low that when the pass wickets are raised at a stage below the top of the main weir sill, head cannot be produced sufficient to interfere with the raising of the pass.

7.—To have the dam of such a type that during the stages of least flow it will conserve sufficient water to maintain a full pool and also afford water for lockages.

8.—To have movable parts of the type which can be maneuvered with safety and ease.

9.—To adopt a type of dam which is the most economical in cost and in maintenance.

The Chanoine wickets and bear-traps appear, from the writer's observations on the Upper Ohio River, at Louisville, on the Great Kanawha, and from descriptions of European dams, to fulfill Conditions 5, 8, and 9 better than any of the other well known types of movable dams.

In regard to Condition 6, provision may be made against such difficulty by placing a 50-ft. bear-trap dam next to and at the same level as the pass sill. In his "Canalization de la Meuse", M. Hans says that the head referred to should not exceed 1 m. The writer believes that it is desirable to prevent its exceeding 1 or 2 ft. In the dams now being built, this condition is fulfilled by placing the sill of both traps at the same level as the pass sill.

Considering the question of tightness, Condition 7, it is believed that bear-trap dams can be made as tight as any other type of dam. Needles, curtains, gates, and, possibly, other types can be made tighter than wickets. With the use of wickets, calculations should show a shortage of water not more than once in 10 to 20 years. The lowest recorded discharge measurement of the Ohio River was taken in 1892, below the mouth of the Muskingum, the discharge being 3 255 cu. ft. per sec. Since this quantity is five or six times as much water as will be required for lockages, it appears that, with cover strips between wickets, sufficient water may be expected nine years out of ten, unless an artificial shortage is produced by the raising of dams up stream. Computations indicate that, with pass wickets 15 ft. high, with weir wickets, 12 ft. high, and with the pass and weir of the lengths proposed, the leakage through the interstices between the wickets will about equal the discharge of the river at a 3-ft. stage. Therefore, during several weeks of each year, needles over the interstices will be necessary.

Considering Condition 5 in reference to the passing of drift, it is found that below about a 9-ft. stage, with no wind, the drift is distributed almost uniformly over the surface of the river. As the water rises above a 9-ft. stage, with no wind, the drift goes to the Ohio or lock shore. The prevailing wind is from the west, and winds of 15 or 20 miles per hour cause nearly all the drift or floating ice to be driven against the West Virginia or abutment shore.

When the bear-traps can be maneuvered readily, they afford as easy a passage for drift and ice as any other well tried type of dam, and a safer one. As their cost and likelihood of getting out of working order increases rapidly with their length, two 50-ft. sections for passing drift were proposed, one in the middle of the river, for use when no wind is blowing, and the other next the West Virginia shore, for use when the wind is carrying drift and ice against that shore.

Conditions 1 to 4 will be considered jointly: At the site chosen there is sufficient room, without widening the river, for 500 ft. of weir, and computations were based on utilizing that width. In making the necessary computations for Condition 1, the most applicable formula is the Chanoine formula used for a similar computation in connection with the Louisa Dam on the Big Sandy,*

$$Q = m\,(L'\,H')\,\sqrt{2\,g\,(h + Z)} \qquad (1)$$

in which,

Q = discharge of river, in cubic feet per second;
m = constant depending on stage;
L' = length of weir;
H' = height of water on crest of dam above sill of weir;
g = acceleration due to gravity = 32.2;
h = height of water, or head, causing velocity of approach of water $= \frac{v^2}{2\,g}$; and,
Z = head due to swell caused by the water being forced through a reduced area on account of the construction of the works.

The values for m in Equation (1) and the following equations were selected from M. Hans' book, "Canalisation de la Meuse," in which all calculations were based on $Z = 0.5$ ft. After the completion of the Louisa Dam the late Mr. Thomas found the value of Z to be about 0.35 ft. when the lock-gates were closed. Therefore, the latter value of Z was adopted, and the correct value of m for the weir for various stages of the river was found, as follows:

0.5-ft. stage, $m = 0.458$
5.0-ft. stage, $m = 0.808$
6.0-ft. stage, $m = 0.811$
7.0-ft. stage, $m = 0.830$
13.0-ft. stage, $m = 0.896$

To determine whether the weir will regulate the upper pool when the water is at a 10-ft. stage, with the top of the weir sill 3 ft. above the top of the pass sill, apply the following values in Equation (1), $m = 0.885$; $L' = 500$ ft.; $H' = 12$ ft.; $h = 0.137$; $Z = 1.7$, the "swell head;" and $Q = 57\,700$ cu. ft. per sec. The discharge of the river at that stage is only 54 600 cu. ft., as shown in Table 3, which gives the discharge, area of section, velocity of flow, and velocity head for the Ohio River at Dam No. 18. Therefore, it appears that with the sill of the weir 3 ft. above the pass sill, the river can be regulated to a discharge equal to a 10-ft. stage, without any spill over the crest of the dam.

* "Movable Dams", by the late B. F. Thomas, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XXXIX (1898), p. 469; or "Improvement of Rivers", by Thomas and Watt, p. 214.

The values of m and Z are only approximate. The value of Z may be considerably in error, but it is conservatively small for such conditions.

TABLE 3.

Gauge.	Quantity, in second-feet.	Area, in square feet.	Velocity, in feet per second.	Velocity head.
0	900	5 210	0.17	.....
1	2 940	6 380	0.46	0.003
2	5 550	7 530	0.74	0.008
3	8 910	8 620	1.03	0.0165
4	13 350	9 940	1.34	0.028
5	19 110	11 030	1.73	0.046
6	26 100	12 310	2.12	0.07
7	33 390	14 420	2.34	0.085
8	40 200	15 820	2.57	0.103
9	47 100	16 920	2.78	0.120
10	54 600	18 350	2.97	0.137
11	66 400	20 540	3.18	0.157
12	73 500	21 810	3.40	0.180

In making the computations for Condition 2, Equation (2) is applied:

$$Q = m\,(L\,H)\,\sqrt{2\,g\,(Z + h)} \qquad (2)$$

in which,

L = length of pass;
H = height of water on crest of dam above the sill of pass; and
Z = swell head.

In these computations a "swell head" of 0.25 ft. is assumed. The problem to be solved in this case is, how high can the weir sill be placed without the "swell head" through the pass exceeding 0.25 ft. Assume that the water is at a 3-ft. stage, or 4.6 ft. above the pass sill, and that the weir is closed. It is found that there can be discharged through the pass, with a "swell head" not exceeding 0.25 ft., 9 186 cu. ft. per sec., whereas at that stage the actual discharge, as shown in Table 3, is 8 910 cu. ft. per sec. Therefore, with the weir sill not more than 4.6 ft. above the pass sill, the "swell head" should never exceed 0.25 ft.

It is found in meeting Condition 3 that if this condition is fulfilled with the water just to the top of the lock river wall (or 16.4 ft. stage) and with the lock-gates closed, it will be fulfilled under all other conditions which are likely to arise. In making this computation, the formula of Chanoine and De Lagrene, used by Maj. Lockwood and the late Mr. Thomas,* for a similar computation for the Louisa Dam, namely,

$$Z = m\,v^2 \left(\frac{S^2}{S_1^{\,2}} - 1\right) \frac{1}{2\,g} \qquad (3)$$

in which,

Z = "swell head," in this case, 0.25 ft.;
v = mean velocity;
S = discharge area of river before construction of works;
S_1 = discharge area of river after construction of works; and
g = 32.2.

* *Transactions*, Am. Soc. C. E., Vol. XXXIX (1898), p. 471.

Substituting the proper values in Equation (3), in which $Z = 0.25$; $m = 1.05$ instead of 1.5 when the "swell head" is reduced from 0.5 to 0.25; $v = 4.34$; $S = 24\,600$ sq. ft.; and $S_1 = 18\,250$ sq. ft.

Using 135 sq. ft. as the discharge area over the top of the piers and abutment, to find the height of the weir sill to give the foregoing area, apply Equation (4):

$$135 + 500\,x + (600 \times 18.0) = 18\,250 \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Solving for x, the height of the weir opening = 14.63 ft. Therefore, 18.0 ft. — 14.63 ft. = 3.37 ft., the height the top of the weir sill may be above the pass sill and not cause a "swell head" exceeding 0.25 ft. Substituting the values of Z and h from Equation (4) in Equation (5):

$$V = \sqrt{2\,g\,(Z + H)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

the velocity of water through the pass at these critical stages is obtained. With the weir closed, and the water 3 ft. deep on the pass sill, the velocity is 2 miles per hour; and when the water is at the top of the river wall of the lock, with a weir sill 3 ft. above the pass sill, the velocity is 4 miles per hour. It is not believed that a velocity of less than 5 miles per hour will seriously interfere with fleets of empty barges going up stream. During construction, in 1908, such fleets passed between the lock and weir coffer-dam against a 7-mile current.

Relative to Condition 4, from experience at other dams, it is known that when the sill is placed too low, there is likely to be serious trouble from sanding.

After considering all these conditions and the development of the bear-trap at that time, the writer recommended that the weir be divided into three parts, as follows:

First.—Next the pass, a bear-trap dam, 50 ft. long.

Second.—Chanoine wicket, 400 ft. long, operated by a bridge.

Third.—Next the abutment, a bear-trap dam, 50 ft. long.

Also, that the sill of the bear-trap next to the pass be placed at the same level as the pass sill, and that the remainder of the weir sill be placed 3 ft. above the top of the pass sill, or 1.4 ft. above low water. These recommendations were adopted, except for the wicket weir which was made 300 ft. long and the pass, which was made 700 ft. long, as ordered by the Chief of Engineers. This resulted in easier and possibly safer navigation for some fleets of boats, but more difficult operation, and, in case of a rapid rise, an increase in the hazard.

In the report of the Lockwood Board* is a diagram of a large coal tow for the upper river. This tow is 130 ft. wide by 696 ft. long. There is also a diagram of one of the largest fleets on the river below Louisville, 312 ft. wide by 1132 ft. long. It is thus evident that in order to operate with ease and efficiency a series of movable dams, the navigation passes must increase in length as the river increases in width. As the weir of Dam No. 18 was designed before any of the others between it and Dam No. 6, its length could not be governed by the lengths of any weirs on the river, except those of Dams Nos. 1 to 6. If designed in order, it is evident that each weir in succession down stream,

* House Doc. No. 492, 60th Cong., 1st Sess., v. 18.

should have the capacity of the one above when fully open, plus the tributaries emptying into its pool.

Wickets.—The location of the exact place for hinging the wickets was studied from both the theoretical and historical points of view, the design of the wickets for all other dams of which plans could be obtained, being considered.

The correct general principle for the hinging of the pass wicket may be briefly stated as follows: The axis of rotation for pass wickets should be placed as low as possible, but at such height that they will never automatically go on swing from head of water alone. Computations for Dam No. 18 indicate that the pass wickets will never go on swing without some force other than the water.

When operated by a bridge, the general principle for hinging the weir wickets may be briefly stated thus: The axis of rotation should be placed so that the wickets will automatically go on swing just at the time the lower pool rises to that height at which the dam should be lowered. In general, the wickets will not be depended on to work automatically, but are designed to work in this manner in case of emergency and for ease in operation. Should a rapid rise occur, and the attendants be slow, or should an unexpected rise occur at night and not be observed, the upper pool may be relieved by the automatic swinging of the weir wickets. Computations for the weir wickets of Dam No. 18 indicate that they will automatically swing at the time the upper pool level rises 6 in. over their tops and the lower pool level reaches a stage of about 9½ ft. above low water.

It would probably be better to hinge the weir wickets so that they would not swing with less than 1 ft. on the crest. However, there is not entire agreement on that rule. After years of operation, the writer believes the hinge point for the pass wickets is satisfactory. For the weir operated with a bridge, the percentage for butt might be slightly increased. For weirs operated with a boat, it is never desirable to have wickets hinged so as to swing automatically. If hinged to swing so easily, they might possibly fail to hold the maneuver boat at stages over the crest. On one occasion a rise with about 18 in. of water on the crest, head reduced to about 4 ft., and acres of heavy drift against the maneuver boat and dam, caused the pass wickets below the boat to swing, dropping the boat over them into the lower pool.

For this study, the writer had computations made which indicated that with very slow current, 6 in. on crest, and with the following height of heads to hinge the weir wickets so they would just swing, the percentage for breech should be as given in Table 4.

TABLE 4.

Head, in feet.	Length of breech, in feet.	Length of wickets, in feet.	Percentage of breech length.
8	5.28	12.92	41.
9	5.48	12.92	42.4.
10	5.71	12.92	44.2
11	5.95	12.92	46.

The designer must consider two forces, namely, the increased pressure against the butts by increased current, and the increased pressure at the top of the chase by increased pressure of the drift or of the maneuver boat. With these conditions known, and his method of operation determined, he is then prepared to determine his hinge point.

After the plans were prepared, the length of the pass and weir wickets and the relative length of the butt and chase were reported for each, as follows:

Pass wickets:

16 ft. $0\frac{1}{4}$ i . long.... { Breech, 8 ft. $2\frac{1}{4}$ in., 51.12% of length. Chase, 7 ft. 10 in.

Weir wickets:

12 ft. $11\frac{1}{16}$ in. long.. { Breech, 5 ft. $7\frac{9}{16}$ in., 43.5% of length. Chase, 7 ft. $3\frac{1}{2}$ in.

The wickets were of long-leaf yellow pine which the writer considers the best wood for the purpose, because it remains true to its original form. In many of the dams, white oak wickets have been used exclusively. The difference in their strength or life for use as a wicket are not material considerations.

The Bebout Wicket.—The only improvement of the Chanoine wicket, which has been used is the Bebout wicket. The first wicket of this type for practical use was installed in the pass of Dam No. 18 in 1914.

Fig. 3 is a view of Bebout and Chanoine wickets at their junction in Dam No. 22, and Bebout wicket horses assembled on their foundation.

The advantages claimed by the inventor for this wicket over the Chanoine are:

1.—It can be raised with ease and without loss of time at any open-river stage up to the top of the wicket.

2.—It can be raised against any differences in pool levels, if sufficient power is applied at the chains; it is possible to raise it against an 8-ft. difference in pool levels with comparative ease and without delay.

3.—It can be lowered under all conditions of river stages and pool levels.

4.—With adaptable appliances, the wicket may be maneuvered from either the upper or the lower pool.

5.—During maneuvers all parts have little exposure to drift, both floating and submerged, whereas Chanoine wickets have great exposure.

6.—When raising wickets, drift cannot, except in extreme cases, obstruct or become entangled in the parts to such extent as to prevent this maneuver.

7.—The sill and foundation of the dam, and the wickets themselves may be completely covered with sand and gravel without serious delay to the maneuvering.

8.—A boat, barge, or heavy drift in striking the top of the wicket will cause it to collapse without damage to boat or dam.

9.—In case of a sudden rise overtopping the wicket, it will either trip or else form no obstruction to navigation.

10.—It is impossible to have the dam caught by a sudden rise in the river, and not be able to lower the wicket.

11.—The sill and foundation of the dam can be washed by raising the butts of the wickets in the same manner as with Chanoine wickets.

12.—The maneuvers are direct and are made in one operation. Raising and lowering may be accomplished in a short time.

13.—The first cost of wickets and foundation should be no greater than that of Chanoine wickets.

14.—For the same factor of safety, the foundation will be materially cheaper; or, for the same foundation, the factor of safety will be greater.

15.—The maneuvers may be performed from the lower pool in case of emergency.

16.—Less equipment and no powerful sinking device is required to sink the wickets.

17.—The ease with which wickets may be maneuvered should reduce the operating costs. There is less wear and tear on the operation of the appliances, wickets, and foundation.

18.—Nine or ten men should be able to operate a lock and dam instead of thirteen or eighteen, as may be required to operate dams with Chanoine wickets.

19.—Since the dam may be raised at any stage of the river, it is unnecessary to allow the lower pool to drop below normal and a 9-ft. stage will be provided without interruption.

20.—Complaints from boating interests will be reduced to a minimum, since no boat should be stranded when dams are raised.

21.—A saving will be effected by the avoidance of a large number of long-distance telephone calls. It will not be necessary to telephone except to give notice of the approach of quick rises. Each lock-master to a great extent can operate his dam independently of other dams.

22.—The wicket is well adapted for use as an emergency gate and could be used as such during the renewal or failure of a lock-gate, and could be made to serve the purpose of a coffer-dam during repairs to locks.

23.—It is possible to dispense with one bear-trap and all the present weirs at the upper river dams, replacing them with a navigable pass-section equipped with the new wickets.

24.—Both the bear-traps and the Chanoine weir could be dispensed with at the lower river dams.

Since the Bebout wickets are maneuvered successfully against heads of 10 ft., or more, a cluster of two or three for passing drift near each end of the pass is such a great convenience, that two or more clusters may eventually be considered essential in the passes of all the dams.

Bear-Traps.—In 1904, when the writer was preparing the plans for the bear-traps for Dam No. 18, the principal traps then known to him were the wooden ones in Dams Nos. 1 and 6, designed in 1898 and 1900, the two steel traps in Herrs' Island Dam, Allegheny River, the three-leaf dam in the Louisville Chute, and the wooden three-leaf dam in the Mississippi River at St. Paul, Minn. All these dams were in service except Dam No. 6, and the engineers directly in charge were believed to consider each suitable for its purpose.

After discussing the merits of each with the District Officer, and after a study of the best papers he could find on the subject, the writer recommended

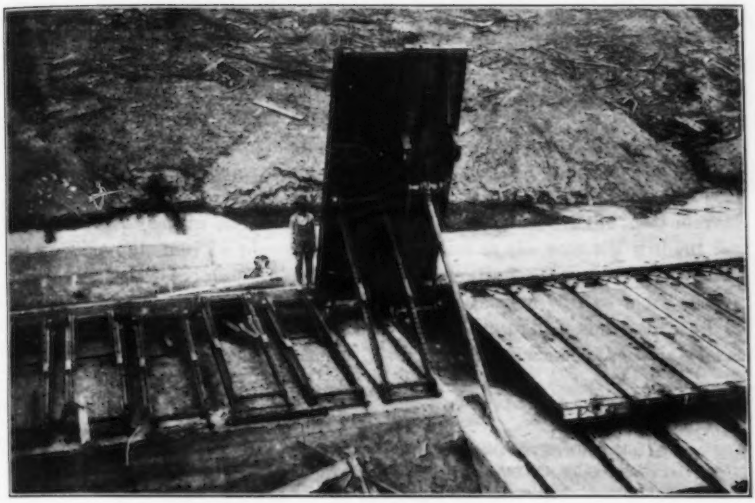


FIG. 3.—BEBOUT AND CHANOINE WICKETS AT THEIR JUNCTION IN OHIO RIVER DAM NO. 22, AND BEBOUT WICKET HORSES ASSEMBLED AT THEIR FOUNDATIONS.



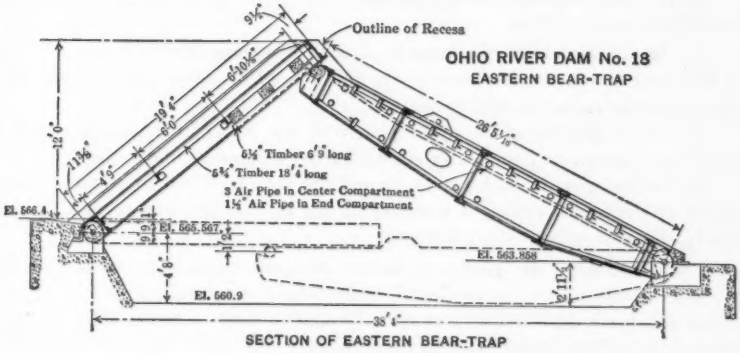
FIG. 4.—VIEW OF COMPLETED WEIR, DAM NO. 18.

steel traps for Dam No. 18, and with the aid of the Junior Engineer, the late C. J. Rannells, Assoc. M. Am. Soc. C. E., prepared the designs and working plans for two 50-ft. traps. Fig. 5 is a cross-section of the eastern or smaller bear-trap.

The traps were completed in 1908 and put in operation in May, 1910, and have always maneuvered with ease, under any conditions of river.

The years of experience with these traps, and the many other traps since constructed for other dams, has demonstrated that for regulating the stage of water in the pools, for ease of operation, and for safety of parts, the bear-traps, in 50-ft. lengths and as since built in 91-ft. lengths, are the ideal movable parts. The sills may be placed at the same level as the pass sill without fear of "sanding up" seriously interfering with operation.

It is believed that the maneuvers of the entire dam will be more efficient and made much easier by placing both traps next to the abutment and the wicket weir next to the pass wickets. With this arrangement, the weir wickets



SECTION OF EASTERN BEAR-TRAP

FIG. 5.

and the pass wickets can be operated by the same maneuver boat, thereby eliminating the bridge which is much more difficult to maneuver and to keep clean of drift than the wickets alone. Therefore, the writer would recommend placing both bear-traps next to the abutment, the 400-ft. wicket weir, with 11-ft. wickets, between the traps and the pass, and making the bank protection below the abutment sufficient to withstand the increased scour.

Care was used in the design to obtain the greatest possible head for acting on the leaves. The girts were made stiff enough to lift the dam with air under only one-third, or one compartment, of the lower leaf, or stiff enough to admit of a deflection of not more than 1 in. when lifted with water alone, the water entering from one pier only and giving a maximum pressure at one end and zero at the other. It was believed that the great deflection under such conditions was the most serious defect in traps previously constructed. The galvanizing of the thin parts of metal and the use of nickel steel were considered; but plain carbon steel was used throughout on account of the cost and the difficulty in galvanizing such large sheets as those used on the skin.

For convenience in cleaning, repainting, or in making repairs, manholes were placed in the lower leaf and in the girts. Wooden water seals were used for closing the gap between the leaves and the masonry. When the central trap was cleaned in 1914, the seals were found to be entirely destroyed, and new ones were not considered essential, since no difficulty had been experienced in operation. In the later designs, metal seals are being installed.

The dimensions of the leaves were as follows:

Westerly, or larger bear-trap:

Length of lower leaf, 34 ft. $0\frac{3}{16}$ in.
Length of upper leaf, 24 ft. $4\frac{3}{16}$ in.

Easterly, or smaller bear-trap:

Length of lower leaf, 26 ft. $10\frac{1}{16}$ in.
Length of upper leaf, 19 ft. 4 in.

All the dimensions are given from center of hinge to extreme end.

Ordinarily, the most severe stresses will occur under the following three conditions of operation:

1.—In the girders of the lower leaf, with low water in the lower pool, and a full pool above, the trap partly up, and a head of water applied with sufficient force to raise the trap to the top.

2.—In the girders of the upper leaf, with low water in the lower pool, and a full pool above, and the water emptied from the recess under the trap.

3.—In the girts of both leaves, should the trap be lifted by water pressure alone, applied with sufficient pressure at one end of the trap to move it and diminishing to zero at the other end.

Computations were made to insure strength enough to withstand the stresses which might be produced under other assumptions not so severe. Since the lower leaf is likely to be subjected to more severe conditions than the upper leaf, the same working stresses were not used. The working stress used in the girders of the lower leaf was 6 200 lb. per sq. in., and in girts 8 000 lb. per sq. in. The estimated cost of the movable part of the eastern trap, 12 ft. high, was $148.38 per lin. ft., and that of the western trap, 15 ft. high, $212.28 per lin. ft. From 1 to 1½ min. are required to raise the bear-traps at any stage with 18 in. of head or more, and they may be lowered at any stage within 1 min. They maneuver with either water or air, or both. Ordinarily, only the water pressure is used.

The widths of the piers from which they were operated and supplied with water are 11 and 12½ ft., respectively. The hydraulic head has always been sufficient to operate the traps when the wickets are up. In submitting the plan for the traps to the District Office, the writer stated, "it appears that, if the mechanical working of the traps now in service was perfect, it would be better to make the traps longer than 50 ft. When the working of the traps has been made perfect, the economical length of any series of traps will depend largely on the character of the foundation, but such lengths will generally fall between 75 and 150 ft." (See page 129.) The standard length for the Ohio River bear-traps is now 91 ft. in most of the dams, and they

are placed between the pass and wicket weir. They are now built and operated with the same confidence of success as lock-gates or other similar structures.

Plan for Lock.—In 1902, when the plans and specifications for Lock No. 18 were submitted to the District Officer, Locks Nos. 1 and 6 had been completed and Nos. 2 to 5, inclusive, were under construction. The plans for Lock No. 18 included the following details which differed from the dams previously mentioned:

1.—Masonry all monolithic Portland cement concrete, instead of cut stone as used in Locks Nos. 1 and 6; or of natural cement faced with Portland cement, or with timber on the chamber walls and coped with stone like Locks Nos. 2 and 5 then being built.

2.—Faces of chamber walls and guide-walls battered ¾ in. in 12 in. instead of being vertical.

3.—The top width of river wall along the chamber, 8 ft. instead of 11 ft.

4.—The flushing of conduits and drift chutes omitted and, instead, each gate recess provided with stop planks and sump for use in annual unwatering and cleaning.

5.—Gate and valve machinery operated by fluid or air machinery instead of steam.

6.—Ten emptying and filling valves instead of sixteen, but with equal capacity.

7.—Each valve operated by its separate engine instead of all being operated by one engine.

8.—Valves rectangular instead of circular.

9.—The pipe well and conduit omitted and, instead, the pipes laid across the lock, descending and ascending the lock walls in open channels in the concrete.

All these features were approved except the battering of the chamber walls and the form of the valves, and, except for the number of valves, have become standard construction on Ohio River dams. Rectangular valves have since been introduced. All engineers in charge of the operation of locks with battered walls, who have expressed their opinions to the writer, prefer the battered rather than the vertical walls. The battered face is not cut so rapidly by the rubbing of boats; it also enables boatmen to walk the gunwales of barges when moored in the lock and, for an equal area of section, it gives a slightly increased factor of stability. The fact, however, that the lock walls when completed were of the vertical type was good reason for not making the change. All the lock masonry is designed for stability as gravity walls; the river wall on the assumption of being completely unwatered and with a stage of the river to the top of the wall; the gate tracks and Poirée dam foundations on a similar hypothesis, and the land wall and guide-walls on the hypothesis of being surcharged retaining walls. They were not designed on the hypothesis of having a full head of water under the entire foundation masonry beds, but it was considered that such a head might occur over small areas of the foundation. Thinner walls of reinforced concrete were considered for the guide-walls, but were discarded on account of the danger from shock by heavy boats or by fleets of loaded barges.

Construction Features of Dam No. 18

Plant and Materials.—In building Lock No. 18, the sand and gravel required for the concrete was taken from the river bed near the site. Coal was obtained by barge from river mines, and nearly all the remaining materials were received on a spur track of a railroad the line of which passed along the top of the bank within 50 ft. of the work. Materials were transferred to the work from storage sheds and yards along the siding by tramway cars operated by men and mules.

The contract for building the dam and abutment was let about a year before the lock was completed. The contractor received materials in the manner previously described, except that his coal and some other materials arriving by rail were received near the West Virginia river bank on a siding about 800 ft. from the abutment, and carted to the work by teams. Materials from the latter siding were transferred to the work by a cableway. The cable was maintained as required by law at a minimum height of 90 ft. above low-water datum. The towers were set well back from the bank, making a clear span of about 1 760 ft., thus securing protection against accident and giving adequate storage and loading space for materials and plant.

The best and most economical method of transfer to be adopted for building any particular lock and dam will be influenced by the economy of extending standard railroad track to the river bank alongside the work, or down into the lock coffer-dam. The use of a cableway is more expeditious than other methods.

It is conceded that every flood over a coffer-dam, when work is fully under way, will cost in actual damage and expense rarely less than $1 000 and usually much more. The cableway is especially advantageous and adaptable to this work, since the cost and time of transfer of the materials and plant from the coffer-dam in case of flood are greater by other methods. Whether or not its use is advisable, depends on the quantity of material which can be moved advantageously. In the writer's opinion, the sand and gravel required for the concrete can be unloaded more economically from the barges, by fixed or floating derricks, and it is more economical to move the concrete from the mixers to the forms by tramway, or by chutes, than to move it and place it by cableway. This economy is due to the fact that buckets from the cableway are not easily dumped in the forms. Also, the sand and gravel must be unloaded from barges moored under the cableway in the navigation channel, which is a dangerous process at stages below 6 ft., and impracticable at stages a little higher, thereby resulting in serious delays to concrete construction. The work advantageously done by a cableway and for which it is especially adapted is the transfer of all materials to their place in the coffer-dam enclosure, and the transfer of plant and tools to and from their place of usage.

Coffer-Dams.—The construction of the No. 18 lock, dam, and abutment required four coffer-dams: One for the lock, one for the abutment in which one section of bear-trap foundation was included, one for 650 ft. of the pass, and one for the weir, bear-traps, and the remaining 50 ft. of pass. For building the

guide-walls of the lock, levees 8 ft. in height were thrown up from material excavated in preparing the foundation.

The proper order for building the coffers is, first, the lock and abutment coffers; second, the pass coffer; and, third, the weir coffer. It is not important whether the lock coffer or the abutment coffer is started first, or whether both are started together. The other two, of course, must follow in succession; but it is important to have the masonry and the bank back of the masonry finished and sufficiently protected against scour, since the latter coffers produce a "swell head" and increased current.

Due to the high sill of the weir, it is essential to build the pass before the weir. On account of the "swell head" and high velocity of current, it is expensive, more or less dangerous, and detrimental to navigation to hold the weir and pass coffers in the river over winter.

The program followed by Government forces and by some contractors on lock and dam construction is to complete the work and remove the coffers in early winter. Some of the slower contractors increase the number of coffers to five, the fifth being made to enclose the bear-traps and adjacent end of the pass. It is unusual for the construction of the lock to be completed in one season; hence, it is customary for the lock coffer to be continued in service over one winter.

For the lock, a crib coffer was built, enclosing the entire lock and lock area as planned and specified by contract. The abutment and dam box coffers were specified with the option of changing, subject to approval by the contracting officer.

In studying the economical height of these coffers, the writer used the hydrograph sheets of daily stages at Marietta, and from them computed the probabilities of coffers varying in height from 10 to 20 ft. being flooded during the working season. With the adopted height of 18 ft., it was concluded that the work would probably be flooded once during the working season from May to December.

In constructing the lock coffer, a channel was dredged with a dipper-dredge, the bed-rock being scraped as clean as possible. The cribs were built in water and sunk on the rock ledge thus prepared. The logs forming the longitudinal walls were barked on the inside, and those forming ties were wrapped with several rings of oakum to retard seepage. To reduce leakage to a minimum all the wall cracks were carefully covered, and selected material was used for filling the pockets. This coffer-dam was built in the spring of 1903 and was in use until the fall of 1904, going through a winter and spring of much high water and a long run of ice from 12 to 18 in. thick. The resulting damage was repaired at a cost of less than $50. Two 12-in. centrifugal pumps easily kept out the leakage, and at stages up to within 2 ft. of the top of the coffer-dam, one pump removed all the leakage. The inside wall was vertical, and stood about 10 ft. from the concrete masonry. The top being so near the permanent work, proved a great convenience for storing materials and offered no obstruction to the transfer of materials from barges and other boats by derrick-boat into the concrete forms or to the top of the wall. This proved to be more economical than the box coffer, with its top 40

ft. or more from the masonry, which is a serious obstruction in the transfer of materials required in the work.

The coffer which enclosed about 600 ft. of the pass, was built in 1905 and was of the box type. It was 602 ft. long inside by 150 ft. wide at the river wall and 200 ft. wide at the outer end, and rested on bed-rock with the top 18 ft. above low water. The upper arm was 20 ft. high at the river wall and 32 ft. high at the outer or channel end. The contractor chose this type at his own option; whereas the writer believes that a crib coffer-dam of the type used for the lock would have been more economical for that part of the pass where the high elevation of bed-rock made necessary a coffer wall only 20 to 25 ft. in height. The pass coffer was poorly built and was wrecked when overtopped by a flood, as shown on Fig. 7. Presumably built to hold out water at 14 to 16-ft. stages, it was effective only to 12 ft., on account of leakage, at which stage three 12-in. pumps were required continuously.

Five separate attempts were made at levee and coffer building for the abutment. Work was first commenced in June, 1905, and continued through the seasons of 1905, 1906, and 1907. The abutment masonry, sheet-pile protection, and the bulkheads extending the seal into the bank behind the abutment, were not finished until November, 1907. The masonry plan showed the foundation bed 22 ft. below low-water level, and the contractor made two unsuccessful attempts to build the abutment within a simple levee made from the light bank material. The third attempt was a rather light box coffer-dam. The two levees and the light box were entirely swept away by the river before the concrete forms were built, and, later, a second box coffer was lost. The fifth and successful attempt followed the construction of a fairly substantial box coffer, protected with round piles and rip-rap.

The weir coffer for Dam No. 18, built by Government hired labor, was placed by having both lower and upper arms run out from shore simultaneously and connected by the channel arm. The timber and metal for the coffer was salvaged from the coffers which preceded it. The bed-rock was from 14 to 23 ft. below low-water datum. The bed on which the sheeting stood was prepared with dredges excavating or refilling as required to level it to 14 ft. below low water. The lumber for the coffer-dam was framed on shore and on a work barge, and erected in position alongside. The width of channel left for navigation between the coffer-dam and the river wall over the pass sill and wickets was 384 ft. and, in addition thereto, the lock chamber was open.

A report made in reference to the coffer-dam while the work was in progress is, in part, as follows:

> Size inside, 590 by 225 ft.
> Height of upper arm, 37.5 ft. at shore to 32.0 ft. at outer end.
> Length of upper arm, 590 ft.
> Pumps required at 12-ft. stage of water, four 12-in. centrifugal.
> Extreme lift by suction, 17.0 ft. } = 38 ft. total lift.
> Extreme lift by discharge, 21.0 ft. }

The difference in the efficiency of the three coffers in holding out the water was so great that it is worthy of note:

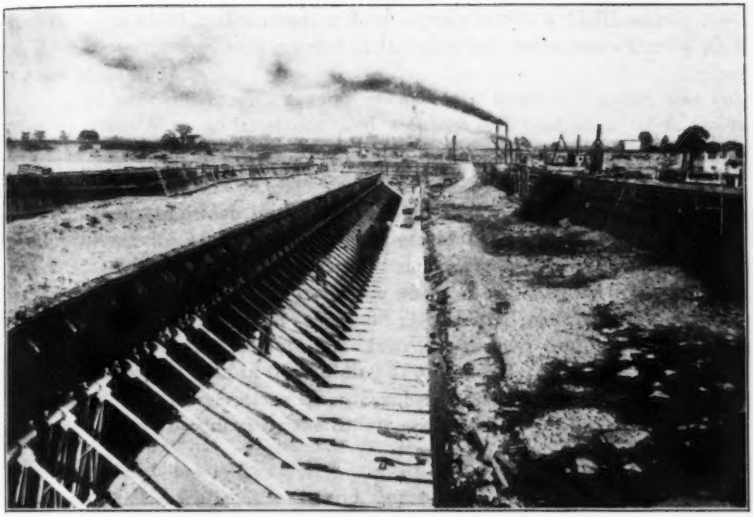


FIG. 6.—VIEW OF PASS MASONRY, DAM NO. 18, OHIO RIVER.

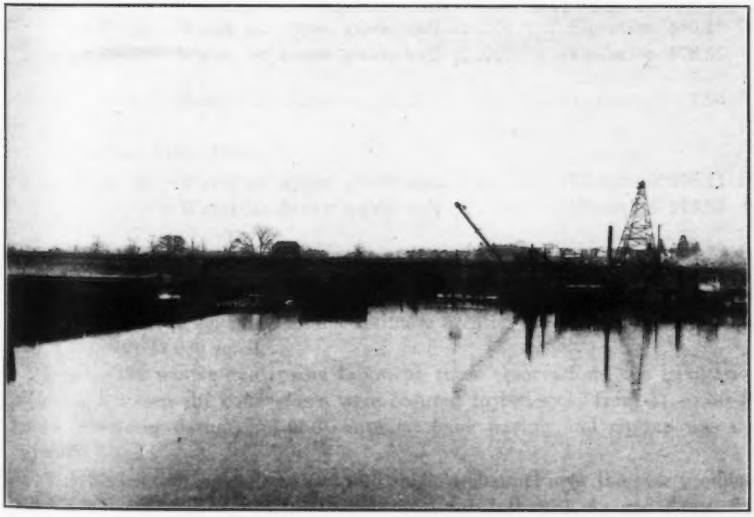


FIG. 7.—DAM NO. 18, OHIO RIVER: VIEW OF COFFER-DAMS AFTER 8-DAY FLOOD, NOVEMBER 30–DECEMBER 10, 1905.

1.—The crib lock coffer set on bed-rock was effective and the work of construction was continued through a flood slightly above a 16-ft. stage.

2.—The weir coffer, with a sump 23 ft. below low water, was effective up to a stage of 12 to 14 ft.

3.—The pass coffer, with a sump about 14 ft. below low water, was only effective from 8 to 10-ft. stages, and actually collapsed at a 9.5-ft. stage, wrecking all the plant inside. Such differences in the efficiency of the coffers had great effect on progress and on the cost of the permanent work.

Two things of great importance in building a coffer-dam are, first, to obtain the service of a designing engineer of experience to select the type best adapted for the particular purpose and duty; and, second, to engage on its construction a foreman or mechanic who can be relied on to close cracks, wrap tie timbers or rods, and select and place filling material so as to reduce the percolation to a minimum. In few construction details does vigilance pay greater returns than in the building of a deep coffer-dam.

Swell Head.—Observations taken October 23d and 24th, 1905, to determine the "swell head" produced by the coffers are, as follows:

October 23d, 1905:

10 A. M.—Water at lower end, upper guide-wall...Elevation 580.89 ft.
Water at lower end, lower guide-wall...Elevation 579.01 "

Head ..... 1.88 ft.

Stage about level with top of 16-ft. coffer-dam and falling.

5 P. M.—Water at upper guide-wall.....Elevation 580.18 ft.
Water at lower guide-wall.....Elevation 578.55 "

Head ..... 1.63 ft.

October 24th, 1905:

9 A. M.—Water at upper guide-wall.....Elevation 578.11 ft.
Water at lower guide-wall.....Elevation 576.89 "

Head ..... 1.22 ft.

The normal regimen of the river at a 16.0-ft. stage before work was commenced was 24 000 sq. ft. The approximate area of the openings, on October 23d, 1905, was 11 080 sq. ft.

During the winter and spring following these observations, the navigation channels between the coffer-dams were scoured to bed-rock, from 17 to 20 ft. below low-water datum, but the abutment bank paving and rip-rap was not injured.

This restriction in such a short and shallow channel over the pass produces the swiftest current caused in the construction of these locks and dams, and is near the maximum which Ohio River boats find tolerable. Under such conditions at Dam No. 33, ascending packets and gasoline boats sometimes used lines to pass the dam and occasionally turned back; towboats cut their

tows in two or more parts during the two winters this condition existed. Under all conditions and positions of the coffer-dams, the maximum "swell head" and swiftest current usually occur at a stage which is about the top of the obstruction and is generally greater on a rising than on a falling river.

Masonry.—Fig. 4 is a view of the completed weir. Fig. 6 is a view of the pass masonry. The 200 lin. ft. in the foreground varies in thickness from 12 to 36 in., the thinnest wicket foundation masonry on the river. The wickets that it carries, have been in successful operation since 1910, which demonstrates the great economy of a rock foundation near the sill level and, also, that a very thin concrete slab well anchored to the ledge is sufficient.

For the pass, it was computed that under extreme conditions there might possibly be an upward pressure of $53 000 lb. against a single wicket foundation slab, 4 ft. in width. The concrete foundation for the pass is 27 ft. wide by 700 ft. long and varies from about 12 in. in thickness at thinnest part of the slab to 15 ft.

Two plans were considered for anchorage, one of which required four 2½-in. bolts for each 4 ft. of the slab, two in the upper edge and two in the lower, with the intervening concrete reinforced with steel rods. The other plan required two 2½-in. and six 1-in. hurter bolts into bed-rock for each 4 ft. of slab, or, in other words, for each wicket. The latter plan was finally approved, except that only two of the 1-in. hurter bolts were authorized to be made for anchors into the bed-rock. Therefore, as built, where the concrete is less than about 6 ft. thick, it is anchored to the bed-rock with four bolts for each wicket, two of which are 2½ in. in diameter and two 1 in. in diameter. The 2½-in. bolts were placed about 4 ft. into bed-rock and the 1-in. bolts about 3 ft. therein. The anchor-bolts for the bear-traps are 2¼ in. in diameter and are placed 4 ft. into bed-rock. When the grout around one of the 2½-in. bolts which went into the bed-rock only 42 in., had stood for less than a day, it failed to move by a lift of more than 20 tons, the greatest pull for which facilities were available.

In laying the foundation, great care was used to make an impervious joint to the bed-rock foundation for the upper third of its entire surface. In other words, the seal between the upper and lower pools was made along and immediately below the upper face of the dam. To make this seal as perfect as possible, all loose shale was removed, and the bed-rock was scrubbed with brooms and water. Thus prepared, fresh mortar was spread over the foundation bed, followed by concrete. In placing the foundation layer, care was used to see positively that no stream of water crossed it, and that any stream which came up through a crevice in the bed-rock was conducted out below the dam without producing any head or upward pressure. Under the lower two-thirds of the foundation, drains were inserted at intervals of about 20 ft. to carry off any water which might seep in. All concrete up to the top of the rock was placed without forms, thus securing bond into all the interstices of the ledge.

The foundation for the chamber walls of Lock No. 18 was prepared by excavating to a depth of 1 ft. below the chamber-floor level, carefully removing all loose rock and cleaning with hose and brooms. Immediately on top of

the ledge thus prepared and drained, grout was spread, in order to make as nearly as possible an impervious joint between the concrete wall and the bedrock. For some locks on the river, a key about 2 ft. by 2 ft. was cut into the rock along the center line of the lock walls.

The superior quality of the structures with the masonry laid on rock ledge over those laid on piles appears to the writer a more important reason for seeking a location with rock foundation and for holding to it when found, than even great differences in cost.

Most of the old cut-stone locks and dams built from 25 to 75 years ago show serious signs of failure. It is entirely possible that most of the locks and dams being built in the Ohio River will be satisfactory for the purpose intended, without change in design, for 100 years or more. Therefore, it appears desirable to take every reasonable precaution to obtain excellency of work.

For that part of the pass foundation of Dam No. 18, which is more than 8 ft. deep, and for all the wicket weir foundation, a plan was prepared for the cellular type, but the bid for the solid type proved to be the more economical. For the bear-traps only the cellular type was planned. In placing the cellular masonry for the bear-traps, drains were used to relieve any head which might otherwise occur. However, entire dependence was not placed on such expedients, but the cellular bear-trap masonry was bolted to the ledge with 2½-in. bolts. There were some indications of the passage of water through the bedrock, principally in the sandstone where anchor holes were drilled, water spurting 3 or 4 ft. from several of the anchor holes for the westerly bear-trap. It was the usual custom to place all the masonry in monoliths about 30 to 35 ft. long, the full width of the wall. In placing the monoliths, the rule was to key alternating sections with from one to three vertical keys about 2 ft. wide and from 6 to 20 in. deep, at intervals of not more than 8 ft., and not more than 6 ft. from the outside surface of the wall. Where the nature of the design would permit, concrete was placed continuously from the foundation bed to the finished top, to prevent a horizontal plane of weakness, otherwise, horizontal keys were placed about 2 to 2½ ft. wide and about 1½ ft. deep. In the wicket pass and weir, care was taken not to permit stoppage closer than 8 ft. from the sill level, thereby insuring a monolith with sufficient weight to hold the wickets. The concrete sections of the bear-traps were placed continuously from bottom to top, except the small parts immediately about the hinges.

After Lock and Dam No. 18 were completed, tests for seepage and hydrostatic head were made by drilling several holes through the lock river wall near the middle of the section and observing the rate of filling and height of rise. In most cases, so little seepage occurred that the upward pressure against the base of the wall was believed to be negligible. Only one hole filled rapidly, a full head from the upper pool being possible. Since the blue shale foundation under that wall is of the same nature and quality as the foundation of the adjacent part of the pass, these tests indicated the possibility of a head against the base of some part of the pass masonry; and the stability of the pass may be due to the anchor-bolts. Experience and observation confirm the writer's belief in the importance and advisability of the use of anchor-bolts.

In August, 1912, after the dam had been in operation for two years, the bed-rock at the toe of the down-stream face of the pass foundation was examined, and no cutting or wear could be detected. Fig. 8 is a view of the completed Lock and Dam No. 18.

Cost of Masonry and Excavation, Lock No. 18.—A tabulation of estimated quantities, contract prices, and amounts, together with the actual cost of the completed work for lock and guide-wall masonry, is given in Table 5. The cost figures were furnished by C. H. Wright, Local Manager for the Contracting Company.

TABLE 5.

Item.	Quantity.	Contract.		Cost to Contractor.	
		Unit price.	Amount.	Unit§ price.	Amount.
Grubbing and clearing, in acres	5.5	$ 25.00	$ 137.50	$ 25.00	$ 137.50
Lumber, in feet B. M.	343 614.0	40.00 per 1 000	13 744.56	33.57	11 535.12
Round timber, in linear feet	35 316.0	0.25	8 829.00	0.17	6 003.72
Common fill, in cubic yards	88 010.0	0.27	23 762.70	0.213	18 746.13
Puddle, in cubic yards	1 055.0	2.00	2 110.00	0.57	601.35
Stone fill, in cubic yards	5 842.7	1.50	8 764.05	1.53	8 939.33
Common excavation, in cubic yards	28 047.9	0.25	7 011.98	0.312	8 750.95
Rock excavation, in cubic yards	11 189.6	2.50	27 974.00	1.294	14 479.34
" " " " *	27.8	3.75	104.25	Separate cost not kept	
Concrete, in cubic yards	20 402.6	4.00	81 610.40	3.50	71 409.10
Concrete cut out of wall, in cubic yards	1.0	5.00	5.00	Cost not kept.	
Paving, in cubic yards†	4 807.6	3.50	16 826.60	5.65	27 162.94
Curbing, in cubic yards†	70.3	7.00	492.10	12.00	843.60
Oak timber, in feet B. M.	2 084.0	50.00 per 1 000	104.20	48.40	100.87
Bolt-holes, in linear feet	415.0	0.50	207.50	0.167	69.30
Iron and steel, in pounds	163 423.0	0.055	8 988.26	0.0445	7 272.33
			$200 672.10		$176 051.58
Vitrified pipe, 8-in., in linear feet	863.0	0.35	302.05	0.045‡	.....
" " 12- " " " "	196.0	0.50	98.00	0.045‡	.....
Galvanized-iron pipe, 3-in., in linear feet	2 404.0	0.60	1 442.40	0.026‡	.....
Galvanized-iron pipe 2-in., in linear feet	2 747.0	0.40	1 098.80	0.026‡	.....
Galvanized-iron pipe, 1½-in., in linear feet	110.0	0.40	44.00	0.026‡	.....
Iron placed, in pounds	106 754.0	0.0075	800.66	.....	.....
Gravel and spawls, in cubic yards	961.4	1.00	961.40	.....	.....
Grand total	.....	.....	$205 429.41	.....	.....

* Rock excavation less in section than 4 sq. ft. and 10 ft. below low water.

† This was the principal work done in 1905; the cost includes a large amount for superintendence and management.

‡ The cost here given is for labor of laying only; the cost of the pipe delivered on the work was not obtainable.

§ The unit cost to the contractor does not include shipment of plant to work, interest on investment, insurance, cost of bonds, deterioration in plant, or general office expense. In this cost was included all labor, superintendence, and clerical accounts at the work, materials and everything of expense, except cost of machinery moved to this work from other jobs of the contracting company. Most of the machinery was provided in this manner. The cost of improvements to machinery, repairs, and maintenance and some new pieces of machinery were included.

Operation of Dam No. 18.

Land for Lock and Abutment.—When the location of Dam No. 18 had been definitely decided on, 5.5 acres of land were purchased at a fair farm valuation, above the top of the bank on the lock shore, including the banks up and down stream to the ends of the guide-walls, together with 3.1 acres on the abutment shore, for use of construction. Long delay in making purchases after locating

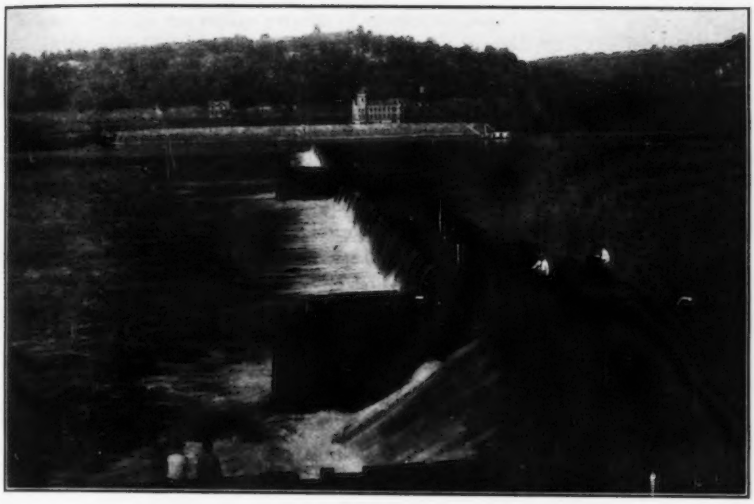


FIG. 8.—VIEW OF COMPLETED LOCK AND DAM NO. 18, OHIO RIVER.

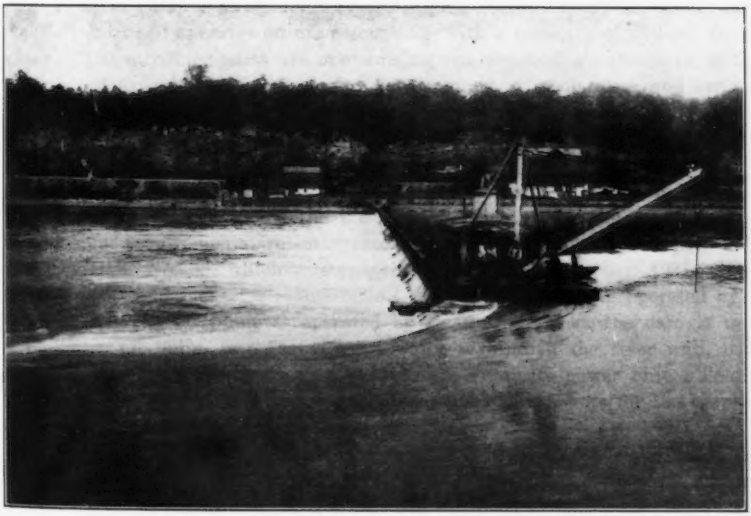


FIG. 9.—VIEW OF MANEUVER BOAT.

the lock sometimes results in paying two or three times the farm value, or resorting to the expensive process of condemnation.

Although the purchase was quite large, compared with the lands acquired for the up-river dams, it was found that more land was needed. In 1910, the District Officer, Maj.* F. W. Altstaetter, Corps of Engineers, U. S. A., M. Am. Soc. C. E., authorized the parking of all the land between the slope of the lock esplanade and well back of the rear line of the lockmen's dwellings, and further authorized the purchase of several additional acres of pasturage.

The writer believes it is wise and to the interest of the Government to arrange such facilities for employees, in order to supply commodities and give remunerative exercise within easy call of duty. These accommodations interfere in no way with their duties, because during many hours they have nothing else to do except wait for boats.

Lock Houses.—Two residences were built having basements of concrete, first story of brick, second story of terra cotta, tile roof with attic finished for storage, furnace in cellars and equipped for hot air, natural gas for light and heat, hot and cold water, with three rooms on each floor, bath on second floor, and laundry in basement. The buildings complete cost about $7 000 each. They are used by the lock-master and chief engineman with their families. All the other employees live in their own homes and come daily to work. Men who do not live within calling distance of a mile are required to have telephone connection with the lock-master's office.

Power Plant.—The operating power plant for the lock and dam consists of two 80-h.p. gas engines, each being sufficient for operation, thus eliminating as nearly as possible any possibility of stopping traffic through the lock. Each is belted to and operates an air compressor with a capacity of 300 cu. ft. per min. The auxiliary parts are one starting engine, four air receivers, with a combined capacity of 1 500 cu. ft., an air pump, a steel tower and tank on the hillside for water supply to the power house, lock, and dwellings.

The lock-gates are operated from a wire line winch with an air-operating engine having a pull capacity of 50 000 lb. in either direction. Each engine is provided with an auxiliary receiver. Where a reliable fuel supply can be obtained, the gas engine is the ideal power machine for the operation of a lock, since requirements are intermittent and, usually, of short duration.

At Dam No. 33, which was opened for service in October, 1921, all the valve jacks and jacks for maneuvering the mitered gates are operated by oil pressure. The oil pump is operated by steam, and when the dam is up, a duplicate oil pump is operated by a water turbine in the river wall, using water from the upper pool, thus providing a duplicate power unit. The use of compressed air is thus reduced to occasional requirements for the bear-trap operation and for signals on the lock walls.

Maneuver Boat.—The maneuver boat is a very substantially built deck barge, 60 by 22 ft., with a cabin 14 ft. wide by 24 ft. long, covering the principal pieces of machinery. The deck when equipped ready for operation stands about 14 in. above the surface of the water and the timber fenders, 6 by 15 in. The principal equipment for the boat consists of one triple-drum reversible

* Now Col., U. S. A. (*Retired*).

hoist engine with three winches, double 9 by 12-in. cylinders and pony drum for slewing the derrick, rigged with ½-in. wire line; one vertical 40-h. p. boiler; one stiff-leg derrick with boom located mid-deck near the bow; one outrigger sheave projecting from the bow for carrying the ½-in. line used in raising and lowering wickets; one steam capstan near the stern with an engine for operating, which carries the line for maneuvering the boat; a boiler-feed pump, a bilge pump, a steam siphon, and a hand pump. The smaller equipment is a wicket raising pole, an extra heavy raising pole, a wicket tripping pole, pike poles, skiffs and oars, 2-in. manila stern line, 750 ft. long, two 1½-in. manila side lines, 100 ft. long, several hand lines ¾ in., 1 in., and 1¼ in., one anchor, and one tarpaulin.

The cost of the boat and the principal equipment in 1909 was, as follows:

Deck barge, ready for equipment	$3 800.00
Engine and boiler complete	1 525.00
Wire lines for derrick	108.14
Wire rope sheaves, etc.	53.10
Spuds, six	170.00
Duplex piston pump, 9 by 5¼ in.	50.00
Orange peel bucket, 15 cu. ft.	499.00
Total	$6 205.74

An electric generator with incandescent and arc lights, with the necessary auxiliary parts, has since been added.

Maneuvers.—The three principal maneuvers of the lock and dam consist of: (1) operation of the lock when the dam is up for the passage of boats; (2) raising of the dam; and (3) lowering of the dam.

The annual force maintained is ten men, which force is supplemented by several additional laborers when the dam is up during the summer and fall. The annual force consists of one lock-master, three enginemen, one dam-tender and five lock-men. Each is an expert at one of the mechanical trades or other useful work, aside from the duties indicated by his position. With spare parts and materials for the machines and structures, this force maintains the lock and dam and appurtenances without additional help, except extra labor engaged locally.

The ordinary maneuvers of the lock for the passage of boats are the same as those required of locks with fixed dams. The usual time for the passage of a boat through the lock is from 7 to 12 min. The time required for the different operations is about, as follows:

Closing gate	1 min. to 1½ min.
Filling or emptying lock chamber	1 " " 4 "
Opening gate	1 " " 1½ "
Departure of boat	4 " " 5 "
Total	7 min. to 12 min.

After the winter floods, or after a summer flood of unusual duration, it requires from 1 to 2 days' time of the entire lock force with a maneuver boat

to remove the deposit from each of the lock-gate recesses, and to clean the gates, engine, and engine pit, and place them in a serviceable condition. It requires a day or more to raise the weir and put the maneuver boat in position to raise the pass. The raising of the pass usually requires from 6 hours to 1 day. Fig. 9 is a view of the maneuver boat raising the pass of Dam No. 1. A wicket prop is being raised with pike pole on butt ready to right the wicket the instant the prop drops over the hurter-step. Since it is desirable both for workmen and navigators of boats to have this maneuver made by daylight, the weir is put in readiness, and the raising of the pass wickets for final closure is usually started in the early forenoon. Raising the bear-traps usually requires only a few minutes by the head of water in the upper pool. The weir wickets are easily "righted up" from the "swing" or raised and put up as desired, if done before there is more than 4 or 5 ft. of head. With from 5 to 10 ft. of head, the work becomes much more difficult.

It is possible to lower the pass in from 2 to 3 hours, but it usually requires from 4 to 8 hours. On very rapid rises, the weir is sometimes lowered at the same time as the pass; but the lowering of the last weir wickets is usually left until the pass is down. The lowering of Dam No. 18 pass usually proceeds from the lock wall to the weir and the raising in the reverse direction. Some dams on the river make these maneuvers in the reverse order.

Maneuvering the Weir.—In 1912, Lockmaster Rowe reported on the maneuver of the dam, as follows:

"Ordinarily, we raise the service bridge with the maneuver boat as the river falls and the time approaches for raising the pass.

"When the maneuver boat is in position at the abutment end of the weir, the boom of the derrick is lowered until the main fall is directly over the chain connected with the section of bridge to be raised. A hook constructed so that it will engage with the bridge chain at any point is used on the main fall. One man is stationed in a yawl, whose duty it is to connect this hook with the bridge chain as low down in the water as convenient. This section of the bridge is then raised with the main fall of the derrick, until the end of the apron is on a level with the top of bridge on pier. A 1½-in. manila rope, with an iron hook on the end, is then hooked into the ring on the end of the apron. This rope passes through a snatch-block on the pier back of the starting point, or on preceding section of bridge, then back through fixed sheave on forward end of boat and is pulled with steam capstan. The rope swings the section of bridge back until the bent is perpendicular. The apron is held up at the proper height with the derrick until in position for dropping down over pins in pier or in preceding section. About four sections can be raised with boat in the same position. The boat lies below the bridge with bow up stream. This maneuver requires seven men: One for operating engine, one for operating capstan, one in yawl for engaging hook, two on bridge for connecting up sections, moving snatch-block, etc., and two on deck of boat for handling lines, moving boat, etc. The bridge can be raised in this manner in one-half the time required for raising it with the winch.

"When the bridge is up, the maneuver boat is then moved over to the pass and two crews are organized so that the weir wickets and pass wickets can be raised at the same time. [Fig. 10 shows a wicket-sinking arrangement for weir service bridge for Dam No. 18].

"In raising the weir wickets with the winch, the duties of the men are about as follows: One man for operating the gasoline engine, clutch-coupling

and foot-brake; one for placing chain on wheel and keeping it from getting fouled in the gear; two for moving and clamping car on track, handling chains, etc. The weir wickets are left on swing until they are all up in that position, and then the same crew rights them up with pike poles after the navigation pass is closed.

"By leaving the weir on swing and both bear-traps down, we rarely have more than 2 ft. of head when closing the pass.

"There is other work to be done aside from that mentioned in raising and lowering the dam, sunken logs, boulders, etc., are encountered, props are found to be out of hurters, hurters have to be cleaned out, etc. When these things are encountered, all the members of the crew work to adjust them. One man who handles the catching hook will sometimes dive down and lift a prop over into a hurter by hand, rather than make the maneuver again."

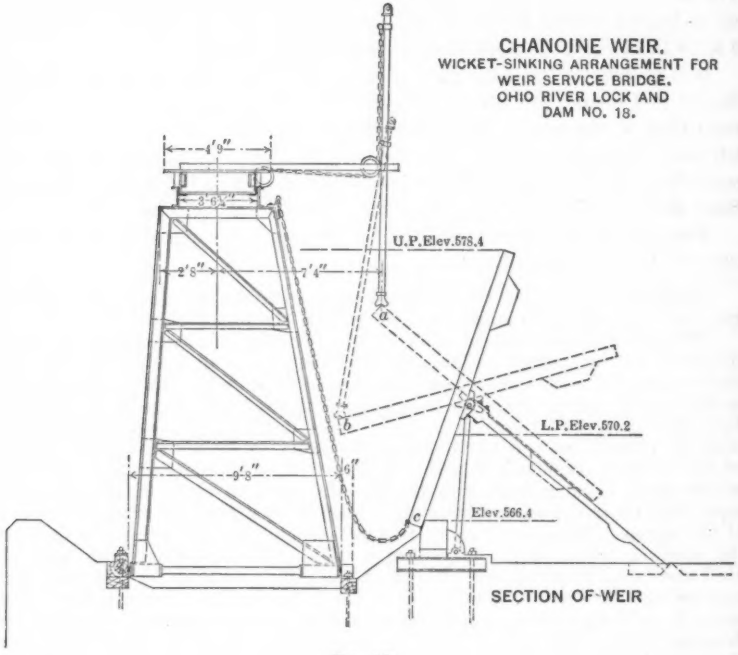


FIG. 10.

Due to the ease of operating the traps and the difficulty of moving the maneuver boat around the traps and their piers, in a dam without a bridge and which has the traps near mid-stream, between the wicket navigation pass and wicket weir, when the dam is up or after raising is started, the lockmen at some of the dams, depend almost entirely on the traps for regulating the pool and neglect to use the weir for that purpose, the function for which it was built.

This difficulty makes it now appear that on those dams having abutment shores of ledge rock, or ledge within 10 or 15 ft. of the sill level, a desirable

location for the traps would have been next to the abutment, thereby placing the weir wickets next to the pass wickets.

Except for the excessive cost of the traps, an ideal arrangement would be to abandon the wicket weir and substitute bear-traps having sufficient waterway to regulate the pools for a natural 10 or 11-ft. stage discharge.

In 1914, the writer had the 15-ft. trap in Dam No. 18 cleaned and repainted. His report of this work is as follows:

When unwatering was completed, a deposit estimated to be about 65 cu. yd. was found in the recess under the two leaves. The deposit on the upper side of the area was largely of silt, but on the lower side near the lower hinge, it was principally gravel ranging from the size of a pea to that of an acorn. Much of the gravel was clean washed and lay 2 ft. or more in depth on the recess floor. In all the pockets of the lower leaf, the deposit was from 14 to 18 in. deep, composed of silt, containing little or no gravel.

On the top skin of the lower leaf, from 60 to 70% of the metal was bare of paint, the remainder being fairly well covered with the shop coat. The under side of the lower skin and the exposed metal on the under side of both leaves were practically bare of paint. On the inside of the pockets of the lower leaf, from 80 to 90% of the metal was covered with paint. After being cleaned, there was slight indication of the metal being pitted by rust.

The two wooden seals were almost destroyed. The lower flange of the lower end of the easterly girder in the lower leaf was bent from 4 to 10 ft. in front of the hinge; the lower flange of one of the other girders was slightly bent; thirteen girder web-plates were bent, and the two lower ones in the easterly girder over the bent flange, were badly buckled; eleven of the cross web-plates were slightly bent; eighteen plates of the lower skin were bent; two bolts near one of the hinges were out, but no sheared rivets or loose rivet-heads were found.

Since the trap had been operated successfully for some time without the water seals, they were not replaced. The damage to the metal work was not considered sufficient to affect seriously its strength, therefore, no attempt was made to repair it.

The approximate cost for all this work was about, as follows:

Labor	$1 158.43
Painting materials	173.01
Lumber	118.48
Supplies, caulking material, etc.	35.00
Miscellaneous	78.57
Gas	85.00
Coal	100.63
Total	$1 649.12

In order to ameliorate the adverse conditions described, the following changes for future plans were suggested:

"(*a*) In each of the piers there should be a pump-well, at least 4 by 6 ft., to admit of using a larger pulsometer. Directly under each of the manholes in the lower leaf, next to the piers, there should be a similar sump.

"(*b*) The piers should be long enough at the lower end to give a plain surface bearing for the end coffer-dam needles. About the perimeter of the lower culvert portals there should be a recess with a plain surface at least 6 in. in width, in order to make the bulkhead more easily fitted and more easily caulked.

"(*c*) The greatest danger from the deposit underneath the leaves is believed to be in lifting or holding the lower leaf above its natural position on the floor, thereby throwing undue strains on some of the girders or creating a short lever arm and bringing excessive pull on some of the hinges. This is especially dangerous where the support under the girders is near the lower end hinges. This may be prevented by placing the upper and lower water passages between the bear-trap recess and the culverts close to the hinges. It is also advisable to make those openings from 3 to 5 ft. high instead of 2½ ft. Should this method not entirely relieve the trouble, in new construction a pipe or culvert laid just below the line of the hinges with an opening alongside each hinge leading from the bear-trap recess to the culvert no doubt could be made effective in washing out any deposit which might occur.

"(*d*) In order to clean the pockets of the lower leaf more easily it is desirable to substitute channels and angles for some of the web-plates. The elimination of this method of bracing and the elimination of half the plates would make the cleaning very much easier."

As the writer was not able to find any record of a trap having been unwatered, cleaned, and painted elsewhere, somewhat better progress could be made on another similar job with this experience.

Maneuvering Pass.—The first sixteen wickets of the pass are raised with the maneuver boat lashed to the pier. After they are up, the maneuver boat is laid alongside them, and the raising proceeds as described by the writer, in his report of May, 1912, as follows:

"The head of the boat is brought up so that the front edge of the bow is within a few inches of being flush with the last wicket raised, the boat being held in place by a stern line to the pier, and two side lines made fast to convenient wickets—one near the bow and one near the stern. With the boat thus prepared and held in position, the lockman casts the raising hook to land on the upper face of the wicket to be raised. After striking the wicket it is drawn along it by a ½-in. wire line wound on one of the drums of the hoisting engine, until the hook engages in the handhold at the butt of the wicket. The wicket is then drawn up until the prop is heard to fall into the step of the hurter. At that instant the lockman disengages the raising hook, and another man with a pike pole forces the butt of the wicket down against the pass sill. At the instant the wicket strikes the sill, a third lockman, tending the stern line, releases 4 ft. of slack, and the current carries the boat forward the width of a wicket. At the same time the engine-man takes up 4 ft. of the head side line while the lockman on the head of the boat maneuvers the side line on the winches of the engine. This movement usually requires from 10 to 15 sec. from the time the butt of the wicket strikes the sill. In general, it is not necessary to operate the spuds in either raising or lowering the wickets During this maneuver the stern spud and upper side spud are raised and hooked up. The lower side spud is left down just far enough to engage the wicket. By so doing, it serves two purposes: The boat is made steady, and as it engages the wicket below the axis, it prevents it going on swing."

In lowering the pass wickets the movements of the boat are a reverse of those described previously. The lockman engages the lowering hook in the upper hand hold of the wicket, and signals to the engine-man to pull on the

line attached to the hook. The instant the lockman hears the prop drop out of the step of the hurter, he disengages the hook, and the wicket, if not obstructed, is carried gently down by the current into position below the sill. At the instant he disengages the hook, the boat is brought in position for lowering another wicket.

The maneuvers of the wickets are often interfered with by obstructions of gravel, boulders, and driftwood. The methods of overcoming these difficulties are only learned by experience.

Water Supply.—The water supply was taken by an air-lift pump from the rear end of the upper gate recess and by sedimentation gave a potable water. At sites where a natural supply is not found, the filter should be constructed at a convenient place in the lock coffer-dam. For Locks Nos. 20 and 33, and for some of the others, filters are constructed in the angle between the upper face of the navigable pass and the river wall.

Signal Lights.—Signal lights are maintained for indicating whether the dam is up or down, and the position of the ends of the river wall and bear-trap piers. The number of lights, their positions, and colors are prescribed by the Engineer of the Light House Department. The lights are carried on standards. Supports of concrete and steel have been tried, but have not yet proven entirely satisfactory. The supports after being submerged and the lights extinguished are a menace to navigation as long as the depth of water is too shallow to prevent boats passing safely over them. During periods of swift current, drift, and ice, it has not been easy to maintain floating supports for carrying the lights. In April, 1915, a passenger steamer hit one of the lower towers on Lock No. 19, sank within a few minutes, and about 17 people were drowned.

Snubbing Posts.—The next greatest menace to navigation are the snubbing posts on lock walls and piers. In March, 1907, a steamer hit the second snubbing post (from head) on the river wall of Lock No. 18 in clear weather, when there was about a foot of water over it, and sank within 15 min. on the head of Neals Island, with several people narrowly escaping drowning. At the time, the water breaking on the nose of the wall clearly marked its position. The light towers and snubbing posts continue to be the greatest source of danger to the navigation of the locks and dams.

Disposition of Drift.—It has been the custom to dispose of the drift by hand or derrick or by letting down one or more wickets or a bear-trap. The writer believes that all big logs and trees should be landed on shore, cut up, and burned, thereby preventing injury to passing boats, or further attention at the next dam below.

The writer wishes to make acknowledgment of courtesies extended by Col. Earl I. Brown, Corps of Engineers, U. S. A., M. Am. Soc. C. E., for access to the records of the first Cincinnati District, as supplemental assistance in the preparation of this paper.

DISCUSSION

THOMAS P. ROBERTS,* ESQ. (by letter).—Referring to the late W. Milnor Roberts, Past-President, Am. Soc. C. E., the author states that Mr. Roberts recommended locks with movable tops on Ohio River dams. The fact is, he never recommended "movable tops" on dams, as such term is understood by river engineers. In 1839, in a report to the Monongahela Navigation Company, he advocated the extension of locks with fixed dams for the Ohio River, such as he was constructing on the Monongahela. Later, in 1857, in a paper published by the Franklin Institute, Mr. Roberts repeated more fully the same recommendation. In 1866, he was made one of the two U. S. Civil Engineers, authorized by Act of Congress, and assigned to the Ohio River to complete the survey of that stream from Pittsburgh, Pa., to Cairo, Ill., and to undertake minor improvements for the immediate betterment of navigation. The chief object of his employment by the Government was to obtain his views on the various schemes, or projects, which had been referred to the Chief of Engineers for the radical improvement of the Ohio.

In his final report, dated April 21, 1870, referring to the dams he was recommending, Mr. Roberts sanctioned the idea of placing in them "chutes" 300 ft. wide, for the benefit of descending coal fleets, thus making navigation practicable for them, even during moderate rises of the river. The chutes were not described in detail in his report, but, to the writer, he stated that chute shutters might be made, say, 3 ft. deep in dams of 6-ft. lift, such as were contemplated by him. The shutters were to be raised by a shaft operated from the lock wall, the shaft turning geared wheels at proper intervals beneath the shutters. The time required to raise them against the water pressure was a matter of indifference, the longer the time, the less the power required. He clung to this idea even after meeting the late Col. William E. Merrill, Corps of Engineers, U. S. A., M. Am. Soc. C. E., his successor in charge of the Ohio River, who had but recently returned from France, where he had seen and was greatly pleased with the Chanoine wickets on the Marne and Upper Seine Rivers. Concerning the project of Mr. Alonzo Livermore who had constructed the locks and dams on the Green River, in Kentucky, Mr. Roberts made no objection to placing a contrivance in the form of a down-river extension of chutes in dams, arranged so that current velocities could be reduced sufficiently to enable ascending steamers to pass through the chutes. Mr. Livermore was to demonstrate the feasibility of his project, which, although theoretically practicable for clear water, might not have been advisable for rivers in which snags and drift complicated the problem during floods.

In an historical review of other projects for river improvement, submitted to Mr. Roberts for his opinion, it may be permissible to refer briefly to some of them. The most important of these projects was suggested, years prior to the Civil War, by Col. Charles Ellet, a distinguished writer on river hydraulics, as well as on railroads and bridges.

Based on ten years' records of rainfall and river discharge at Wheeling, W. Va., made under his personal supervision, Col. Ellet demonstrated that

* U. S. Engr. Office, Pittsburgh, Pa.

a 6-ft. minimum to 9-ft. maximum stage could be permanently maintained on the Upper Ohio, provided the excess of flood water on tributaries could be stored in reservoirs. This project was so ably set forth by Col. Ellet and Mr. Elwood Morris, as to attract much public attention and was even favorably considered by Congress which was about to appropriate a large sum for starting surveys for reservoirs, when Mr. Roberts, almost alone in his opposition, interposed with his pamphlet entitled, "Practical Views on Ohio River Improvement". He showed that, in 1847, to have had full control of storm waters, the reservoirs above Wheeling should have had a capacity of more than one trillion cubic feet. His strongest point was that the records indicated the probability that some of the more important reservoirs would have been full when certain floods occurred. Other river engineers, including Col. C. McD. Townsend, U. S. A. (*Retired*), M. Am. Soc. C. E., formerly President of the Mississippi River Commission, have since demonstrated that, as great river floods frequently originate from comparatively limited areas, for instance, such as have created very high floods at Cairo, such floods might be aggravated by waste waters from reservoirs at the head of the Ohio, etc. It has been proved that to reduce materially the height of great floods at Pittsburgh, restraining reservoirs should be as near as possible to the city, for the flood-producing storms reaching Western Pennsylvania are only rarely more than 90 miles in width of excessive precipitation. However, on rare occasions, Nature doubles, or even trebles, what is termed her excessive precipitation, as appeared in 1913 over Northern Indiana and Ohio, just covering about one-eighth of the Allegheny River water-shed. That one-eighth, however, with a precipitation of from 7 to 9 in., produced a flood which at Pittsburgh, 100 miles below, was greater in discharge than any preceding flood developed from more than triple the same storm area. Had the 1913 storm moved eastward only 80 miles farther, careful study indicates that, at Pittsburgh, the flood would have been at least 15 ft. higher than the record flood of 1907. It is useless to attempt to estimate the damages which would have resulted from such a flood to the towns, bridges, etc., above Pittsburgh. The 200 acres of low land embraced in the business section of that city, with currents of from 8 to 10 miles per hour, of from 12 to 14 ft. in depth, through its streets, can be imagined if not described. What would have been left for salvage of the city would have been chiefly steel beams, protruding from a great area of gravel, building materials, etc. To talk of restraining reservoirs for such floods, at least for the area above Pittsburgh, is waste of time. From the evidence of the topography of the Allegheny Valley, glacial drift remains, etc., no such flood has occurred at Pittsburgh since the last flood waves of the glacial epoch passed along the Upper Ohio Valley not less than 10 000 years ago.

Wrong impressions of the transportation capacity of the Ohio River, when finally improved, should not be perpetuated in the publications of the Society. It is of the first importance that Congress should not vote away public funds on projects that will not adequately benefit the public interests. More and more it is being demonstrated that without the utilization of the inland waterways to cheapen transportation to the seaboard, all hope of a

world-wide extension of commerce must be abandoned. The railways now staggering under the loads imposed on them, would be greatly relieved if some, such as coal, and other items of low intrinsic value, could be taken from them.

It is unfortunate that the river engineers of the United States competent to portray the advantages of waterways are, in a way, debarred from promulgating their views, but must usually confine themselves to the engineering duties assigned them. On the subject of the real potential capacity of the Ohio when improved throughout its entire length, Mr. Hall writes as follows:

"The capacity of the river for package and barge freight, when the improvement is completed, would easily be 25 000 000 tons annually and possibly double or treble that quantity. However, during the months the dams are up, the lockage capacity would be much below that rate."

From such a statement the inference is that locks 600 ft. long by 110 ft. wide (the same width as provided for the Panama Canal), and 9 ft. deep on gate-sills, would be "much below" a capacity of 25 000 000 tons annually.

In the Pittsburgh Engineer District there are included ten locks 600 ft. by 110 ft. on the Ohio, three locks and dams on the Allegheny River, and fifteen locks and dams on the Monongahela River. Six of the dams on the Monongahela, next in order above Pittsburgh, are provided with twin parallel locks averaging a lift of about 9 ft., with available chamber dimensions of 360 ft. by 56 ft., with limiting depth, at one lock, of 9.2 ft. on gate-sills, up to, and including locks, at Dam No. 6. Yet, during 1920, with practically all the traffic being compelled to pass through the locks, there was accommodated 24 000 000 tons. The lock most used was at Dam No. 3, where for three months during 1920, the traffic was confined to one lock chamber. During this period of three months, there were passed through the single lock in one day 93 000 tons, with not quite all the time available for locking utilized. The tonnage was nearly all down stream, while empty fleets (which might as well have been loaded) passed upward. Upward bound fleets meeting descending fleets have both been passed through Lock No. 3 in 35 min., although 40 min. are usually allowed (single steamers, of course, in both directions in considerable less time). Had this traffic through a single lock at Dam No. 3 been maintained for a year of 280 days, it would have amounted to nearly 26 000 000 tons.

Only stern-wheel boats equipped with balanced rudders are used on the Monongahela. When backing their wheels, these boats develop an enormous steering power, and are capable of doing such a business safely in the narrow confines of lock approaches. Collisions of vessels or serious damages to lock-gates are rare occurrences on the Monongahela. "Tunnel boats", or independent double-screw boats, will probably never be built for the rivers about Pittsburgh, as they are all deficient in steering power. Pittsburgh is the seat of inland waterway transportation, more than equalling the present tonnage of the entire Mississippi and its tributaries below the city. Here, also, were made, at much cost, experiments with bear-traps, Chanoine wickets, roller lock-gates, etc., leading up to further improvements elsewhere on the river. There is no better school than "the school of experience".

The Monongahela locks will now be compared with those built, or in process of construction, on the Ohio. Fleets of 9 000 tons have been frequently passed down the Ohio through Locks Nos. 1, 2, and 3, which locks have the same dimensions as all locks on the river below to Louisville, Ky., 600 miles below Pittsburgh. These fleets were passed through these locks, usually in 20 min., and ascending fleets meeting descending fleets, including the time required in filling the lock chambers, were passed in 45 min. Considering adverse winds, and contingencies of freshet currents approaching locks, it is considered safe to say that fully 10 000 tons per 50 min. could be passed down the river, and a theoretical tonnage of nearly equal amount might be passed up. Therefore, 50 min. would permit of twenty-eight down, and twenty-eight up, lockages per diem. Ignoring up-stream tonnage and restricting down lockages to 8 000 tons would make a possible down tonnage of 224 000 tons daily, as compared with the 93 000 tons daily for a single lock on the Monongahela, previously referred to.

The writer has been in charge for more than 20 years of Monongahela locks and dams and is fully aware of the likelihood of locks being closed for repairs, necessity for which at times may put a lock out of commission for extended periods. The Ohio locks, however, have an important advantage over those on the Monongahela, where there is no open-river navigation, in the fact that, on the Ohio, repair work is often practicable during the periods of open-river navigation, or when the locks would not be used by passing vessels. Doubtless, however, Ohio River dams, in due time, as necessity arises, will be provided with second locks, such as are already constructed at the Emsworth Dam on the Ohio, 9 miles below Pittsburgh.

As open-river conditions will be maintained longer on the Ohio River below Wheeling, 90 miles below Pittsburgh, than above that point, there will be a greatly lessened dependence on locks below in the river to Cairo. With open-river conditions, the capacity of the Ohio for traffic, of course, would be exceedingly great, but one must not be unmindful of the fact that, for a regular annual traffic on the Ohio, the worst low-water conditions should be assumed, for the traffic is not dependent on weather conditions, but comes from human needs—"the law of supply and demand." It can be depended on that the capacity of the river when the growth of traffic demands it, can be made considerably more than 100 000 000 tons per annum passing any given point, or more than four times the present annual traffic of the Monongahela River with its small locks and no open-river navigation.

At the risk of unduly prolonging this discussion, the writer urges attention to an important topic raised by Mr. Hall in the last paragraph of page 97. It concerns, as he says, the "fallacy of free rivers" improved and maintained by the Government and left for "Tom, Dick, and Harry" to rival each other in picking up package freight and cutting each other's throats with lower and lower rates, until finally no profit is possible. At present, the Ohio is, to a considerable extent, only a playground for the owners of small locally owned boats engaged in short-distance transportation. Under such circumstances, no responsible companies will undertake to construct on a large scale, first-class

terminals and operate steamer and barge lines on schedule time, and never will, until something is done under the auspices of the Government. The writer in saying this does not wish to discredit several small companies recently organized to place steamers on the river above Cincinnati to Pittsburgh. They are doing well and their business is increasing.

In time, there must come on the Ohio, steel barge lines towed in fleets for long-distance transportation, that is, freight towed in fleets separate from the miscellaneous light traffic possible for fast passenger boats, before traffic worthy of attention will be developed. The barges will have to be loaded at the mills and mines in 500 to 1 000-ton lots, otherwise, the old low ton-mile rate will never be revived. Rivers in this respect are in no way different from railways. However, with rivers, no maintenance of "roadbed" is necessary.

The writer, several years ago, stated in a published article that the freedom of navigation on the Western rivers had led to almost the entire extinction of traffic on them. Nevertheless, it is best that the rivers be left free, if for no other than physiological reasons. To charter companies, would lead to the development of animosities fraught with mischief.

To revive interest in the rivers, the writer proposed a plan somewhat as follows: First, the appointment of a commission by the Government directed to visit every important commercial or manufacturing community along the rivers (say, for a trial section, the Ohio between Pittsburgh and Louisville, 600 miles), the navigable tributary streams to be included, to obtain information as to their commercial or manufacturing resources, tonnage traffic, etc. Second, that the commission be authorized to meet boards of trade or prominent business men and inform them that for the cost of modern terminals and railway connections the Government would subscribe 51%, provided the citizens of the locality subscribe 49% of said cost, the Government guaranteeing 3½% on all private investments (approved by the commission, and, later, of course, indorsed by the National authorities). Third, that boat lines, steamers, barges, repair yards, etc., be organized in the large cities on the same basis, no profit in the operation of boat lines or terminals to be in excess of 3½% on the total investment of capital including maintenance, renewal of plants, etc.; any further profit desired by private subscribers to be found only in the extension of their business by trading with distant markets secured to them by the boat lines.

The writer's conviction was, and is, that this is the best and surest method of interesting vast numbers of people in the benefits of inland navigation. Any small companies desiring to compete with the semi-national boat lines should be allowed to do so, except that they should pay proper charges, where they utilize the National terminals. All the Government desires is to develop the usefulness of the rivers. If private boat companies can be organized to do business, on a reasonably large scale along the rivers, at rates lower than the official rates, then the "official" boats can be withdrawn from the trade and utilized elsewhere. Private subscribers to terminal landings, where such would be abandoned for lack of income, should be refunded.

With this proposition nothing would be done by the Government unless it was amply assured by the data secured by the investigating commission.

EARL I. BROWN,* M. AM. SOC. C. E. (by letter).—The improvement of the Ohio River for navigation purposes, under the present project, is a problem of intense interest to all engineers who have been connected therewith. Literature on the subject describing the locks and dams and the methods of construction used to meet the special problems that arise in connection therewith, is scarce, and Mr. Hall's paper is a valuable contribution to it.

As explained by the author, the project for improvement has been carried forward progressively from Pittsburgh, Pa., toward the lower end of the river, to such an extent that practically all the locks and dams between Pittsburgh and Louisville, Ky., either have been completed or placed under construction, as well as several between Louisville and Henderson, Ky. The experience gained in the construction of these locks and dams has been such as more or less to crystallize and systematize the methods of construction for these structures. The methods described by the author for Dam No. 18 may be taken as representative of the later practice in the construction of locks and dams on the upper part of the Ohio River as far down as Dam No. 31, at Portsmouth, Ohio.

Practically all the dams from Pittsburgh to, and including, Dam No. 31, are founded on rock. This fact simplifies the construction on account of the inability of the current to erode the bottom of the river when the bed is contracted by the coffer-dams utilized for construction purposes. In several instances, on dams below Dam No. 31, constructed on sand and gravel foundations, extensive erosion of the banks and bed of the stream has occurred during freshets, due to the presence of the coffer-dams. This has resulted in difficult and expensive work in filling the eroded area and in replacing damaged parts of the coffer-dam and other structures.

The quantity of sediment carried by the river in times of flood also varies greatly in different parts of the river, increasing from the head-waters downward. In the upper half of the river, construction work was not greatly hindered by deposits in the working area within the coffer-dams during floods, but, in the middle and lower sections, these deposits become very great and cause considerable loss of time and great expense for their removal after the freshets subside and before construction operations may be resumed. Dam No. 48, near Henderson, Ky., is the lowest one of the series on which any work has yet been attempted, and its history has been one of great difficulties encountered and unexpected cost from shifting sand and damage by flood and deposits.

These conditions have led to a reconsideration of the project, with a view of meeting these increasing difficulties by some modification of the plan. Studies have been made with reference to the possible elimination of all locks and dams below Dam No. 49 and a substitution therefor of a system of improvement by regulation. It has been pretty well established that a system of regulation by open channel work below Dam No. 49 is feasible, but that the expense of such a system of regulation would exceed the cost of locks and dams above the mouth of the Tennessee River at Paducah, Ky. Below

* Col., Corps of Engrs., U. S. A., Wilmington, Del.

Paducah, it is probable that improvement by regulation would not be more than 50% of the cost of improvement by locks and dams. This is due to the fact that this part of the river, about 47 miles in length, is affected by back-water from the Mississippi River at Cairo, and the low-water discharge is practically doubled since the Ohio receives the discharge from the Tennessee River, the low-water discharge of which is practically equal to that of the Ohio. It may be stated, therefore, that these studies indicate that the project might be modified further to advantage, by the elimination of Dams Nos. 53 and 54 in that section of the river between Paducah and Cairo.

By reason of shifting sand bottom, broad channel of the river, lack of rock foundation within reasonable reach of the surface, and small low-water discharge, the construction of dams from No. 48 downward will be difficult. Moreover, back-water from the Mississippi River extends its influence up the Ohio River at low water to great distances, practically to the location of Dam No. 49.

As the coffer-dams in which these structures are built cannot with safety be constructed to exclude freshets of more than 16 to 18 ft., the number of working days possible during the year depends on the number of days the river, at the site of the dam, is at and below these stages. In the upper part of the river, where construction has heretofore been carried on, the number of days the river is at and below these stages is relatively large, averaging well over 100 days per year. In contrast, an examination of the records of the stages of the river at Cairo, near the location of the proposed Dam No. 54, shows that the river is at and below an 18-ft. stage only about 60 days in the year, leaving only about 2 months available for active construction work. The present types of coffer-dam, therefore, would become increasingly inefficient as the work progresses down stream.

In those locks that are not founded on rock, certain modifications of features, previously considered to be standardized, have been found to be desirable. To prevent a leakage under these structures through the sand and gravel foundation, it has been found necessary to provide a cut-off wall of steel sheet-piling extending continuously across the river from the lock to the abutment and penetrating the bottom to a depth of from 30 to 40 ft. This cut-off wall is also carried entirely around the chamber of the lock so as to exclude leakage into the chamber. With this continuous cut-off wall, it has been found possible to omit the concrete floor of the lock, which has hitherto been provided, and to substitute therefor a layer of broken stone about 2½ ft. thick, at much less expense. Most of the locks heretofore constructed have been provided with a Poirée dam at each end, for the purpose of unwatering the lock when making repairs to the chamber and gates. It has been found feasible to omit this, utilizing instead a short section of ordinary box coffer, which can be placed in position in about the same time as the Poirée dam and which reduces the first cost of the lock by several thousand dollars.

Most of the locks heretofore constructed have been provided with rolling gates, which type was developed to suit local conditions on the Upper Ohio River. On the lower part of the river, many objections have developed to the

rolling gates, and it has now been definitely decided to construct no more dams with this type of gate, but to use instead the ordinary mitering gates. The principal objection to the rolling gate is the fact that it requires one or two weeks at the beginning of each season after the freshets subside to place the gates in commission, by reason of the difficulty experienced in removing deposits of mud and sand which accumulate in the gate recess. It has been found impracticable to seal these recesses sufficiently to exclude the deposits of fine mud and sand, which are carried by the winter floods and which increase greatly in quantity with distance down stream. This trouble does not occur with the mitering gates, and the expense of construction is not greatly different from that of the rolling gates.

In the operation of this system of locks and dams, much trouble has been experienced by reason of inability to raise the dams at stages higher than about 9 ft. This results in the impounding and holding back of the discharge of the river in the pools of those dams that are raised, with a consequent rapid falling off in the discharge of the river at points lower down. So great has this effect on the discharge of the river become, that at times much delay and obstruction to navigation has been experienced at and below Louisville, by reason of the manipulation of the dams at and above Cincinnati. As the number of dams in operation increase, the effect on the lower part of the river will become more and more marked, and, at times, may result in an almost complete absorption of the discharge of the river, if great care is not exercised in the raising of the upper dams.

To meet this difficulty, a comprehensive scheme of control of operating the dams will have to be devised, so that constant reports as to the stages of the river and variations of discharge may be received at a central point, at which will then be determined the proper time for the raising and lowering of all dams, thus regulating them so as to affect the discharge in the lower parts as little as possible. A telephone system is being provided, connecting all dams to a central point, which will probably be Cincinnati, and that will be the point from which this centralized control will be exercised. After some means can be devised whereby the dams can be raised at stages above 9 ft., much benefit will be gained, and it is understood that attempts are now being made to devise some method of raising the dams at higher stages.

On several of the locks and dams, experiments have been made in the use of steel sheet-piling for the construction of coffer-dams, with some degree of success in the matter of increasing the number of days on which construction operations can be begun. This experiment is being carried out most fully at Dam No. 34. It is hoped that additional information covering this mode of construction will soon be available.

William W. Harts,* M. Am. Soc. C. E. (by letter).—To those members of the Profession who are interested in the navigation of inland rivers, the author's painstaking and excellent paper should be welcome as a valuable addition to their information on this subject.

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Much has been written on the engineering projects undertaken on the Ohio River during the last fifty years, but it has been mainly since the adoption, in 1875, of the present method of slack-water by means of movable dams, that this river has acquired a great engineering interest. It may be stated positively that the present slack-water system of the Ohio River is unsurpassed anywhere in the world, for suitability to local conditions, practicability in operation, economy of cost and maintenance, and success in meeting physically the needs for which it was adopted.

For many years, much attention has been devoted to the details of this system, and examples of successful practice elsewhere in the world have been examined and studied and often combined with new inventions in a project for this stream, which now challenges the admiration of engineers generally for its excellence and success.

The east and west direction of the course of the Ohio, so fortunately parallel to the great east and west freight movement in the area south of the Great Lakes, together with the large deposits of coal along its head-waters, often on the very banks of its tributaries, gave an enormous importance to the navigation of this river in earlier years and stimulated the need of better channels and depths as the pressure for fuel increased in the West, and as the size and unwieldiness of coal fleets increased.

Already, some of the tributaries had been made the subject of engineering attention by the States through which they passed, and their steep slopes had been overcome during these earlier years by slack-water projects, using locks and fixed dams. Through these developments the coal on some of these tributaries could be brought out more easily during the year and held in pools ready to float down the Ohio in huge fleets on the spring floods, to supply the growing markets of the Lower Ohio and Mississippi Rivers.

This important traffic in coal, thus commenced, grew in later years to an enormous volume, reaching, in 1900, nearly 15 000 000 tons. It was only natural that it should have been the early aim of the Government engineers to use the well-known slack-water system, already applied to some of the tributaries, for application also to the Ohio River, but until the early objection to the fixed dams of the tributaries could be overcome nothing could be accomplished.

The Ohio is subject to extreme floods. Sometimes, as much as 70 ft. in range is recorded, and great injury and loss invariably result to the towns along the banks. Nothing could be allowed in the bed of this stream that would be likely to increase this range, and in the public mind fixed dams were objectionable for this reason. Large quantities of sand and mud are carried down at each freshet, which, it was feared by some, might fill the channels if fixed dams were used and thus ruin the existing navigable depths, and, furthermore, the pilots of the towboats which guided the great fleets of coal barges down the Ohio at high stages, would not consent to any obstruction that might make the management of these fleets more difficult at open-river stages. A movable dam of some type was believed to be necessary to meet these objections.

Accordingly, search was made everywhere for types, and the following were finally found to be the best suited to the condition, namely, wickets, such as the

Chanoine, needles, such as the Poirée, and bear-traps, an invention of the lumbermen who used this kind of dam to create an artificial wave to float out lumber rafts on the head-waters of streams of small flow. After trial on one of the tributaries, the needle type was abandoned for use on the Ohio, and bear-traps were adopted as a regular device for controlling levels.

The first dam was of the wicket type, of low lift, and was authorized by Congress in 1875 on the recommendation of the late Col. W. E. Merrill, Corps of Engineers, U. S. A., M. Am. Soc. C. E., who, with much justice, has been called the "Father of the Ohio River Navigation". Much study since the construction of the first dam has produced the present type of structure, and in 1910 the canalization of the entire river was authorized by Congress at a cost then estimated at $63 000 000. Before that time, the engineering features had been mainly standardized in practice, but new improvements were eagerly sought. The author's description of these standard designs, as applied to Lock and Dam No. 18, are clear and instructive. It is believed that a further discussion by him of his reference to the economics of the entire project as viewed under present conditions, would have been enlightening and profitable.

The enormous development of the steel industry in the Pittsburgh region and the corresponding increase in the demand for coal for smelting purposes have led to the more perfect control of the coal measures of the Western Pennsylvania regions by the steel companies, with a view to conserving the present stock. This has succeeded so well that a large volume of coal no longer seeks a market by the Ohio River; this export has been almost stopped, and coal is even imported to the Pittsburgh region from the West Virginia fields. Furthermore, the development of the Alabama coal mines and the discovery of oil in Oklahoma have reduced the fuel demands for Ohio River coal in the Lower Mississippi Valley. The low cost from the newer mines of Western Kentucky and Southern Illinois also reduces the demand for Pittsburgh coal. All these influences have had a marked effect on the commerce of the Ohio River of late years. In 1900, the commerce, principally coal, amounted to about 15 000 000 tons, whereas in 1920 it had fallen to about 5 000 000 tons. Thus, it appears that in about 20 years the commerce had dwindled to about one-third its former proportions, notwithstanding that, in the meantime, many troublesome shoals had been eliminated by the new locks and dams which were completed from time to time, thereby materially increasing facilities for navigation, as may be seen from the fact that thirty-three of the proposed fifty-four locks and dams had been completed by the latter date.

If the traffic diminishes, the overhead costs, being more nearly fixed, increase relatively, but are still a proper charge against the costs of water transportation. The main interest which the public, as a whole, has in works of this character, is in the saving that may be made over other kinds of transportation, in money or its equivalent. On no other basis can works of this class be justified economically. The overhead charges for interest on the first cost, depreciation, and other annual charges, together with the cost of maintenance of the channels by the public, is undeniably a part of this cost of transportation and must be added to the shipper's cost of water haulage to

ascertain whether the actual cost of transportation by water is less than by other means. A short computation should reveal the facts in this regard. If there is thus shown to be no saving or a loss in developing transportation by water rather than by other methods, all features considered, the enterprise cannot be classed as an economic success and its value as a public venture may be negative. In such a case, may not the expenditure of public funds on such works be criticized as of questionable propriety? Are there any important considerations other than an economic justification that may warrant the great outlay for such works?

It seems plain that engineers should never permit themselves to be lured from the true economic solution of such transportation problems by the blandishment of local commercial organizations which sometimes desire large sums to be spent in their localities with only a minor regard for the National service or the economic success of the project; nor should engineers permit themselves to be misled by glittering speculative estimates of future traffic that a more careful study would show could never develop.

The supreme test of the public value of any inland waterway must always be an economic one. Can the actual ton-mile cost to the shipper of hauling by barge and towboat, or by other similar means, when added to the ton-mile cost of interest on the first cost, depreciation, and maintenance of the waterway now borne by public taxation, effect, when combined, a saving over other means of transportation? How many of the inland streams, on which extensive navigation projects have been built, can now meet this test?

The low credit of the railways and the resulting cramping of their development of late years by the present public policies, may have a serious effect in the future, when a new prosperity shall follow the present depression. When that time comes, many years and much money may be necessary before the companies can replace their railways on a basis sufficiently broad to handle the volume of traffic presented. At that time, water transportation may perhaps be considered, to supplement other methods, and many of the waterways may then arrive at a new prosperity and public usefulness. How permanent and extensive this might become is, of course, subject to the usual economic laws from which there is no escape.

L. M. Adams,* M. Am. Soc. C. E. (by letter).—The author's interesting paper can be read with great profit by any one who may have similar works to build in a river subject to sudden and irregular flood stages, such as the Ohio.

It was the writer's good fortune to have considerable to do with the design of the later locks and dams on the Ohio River, and it is his recollection that the controlling idea in adopting the Chanoine type of movable dam for the Ohio River System was on account of the peculiar method, so much in vogue, of making up, during ordinary river stages, great fleets of loaded coal boats, which moved down stream on subsequent rises of the river. These fleets were so large that the stern-wheel towboat which controlled them, could do no more than steer the floating raft and, where conditions warranted, return to the Pittsburgh Coal District with the empty barges many weeks later.

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The custom had become established of handling fleets of barges in this manner and the mere suggestion of placing fixed dams in the river, which would require the fleets to be broken up to pass through the locks, brought violent opposition which could not be overlooked.

As practically all the tonnage movement was down stream before and during the period the movable type of dam was adopted, the question arose at the time, whether it would not be more desirable to build fixed dams of comparatively low lift, say, 6 ft., with the idea that during fair sized floods these low fixed dams would be drowned and the coal fleets could pass down the river in the accustomed manner, special care being used not to start until the disturbance at the dams had become a safe minimum. However, the necessity of slightly increasing the flood heights with consequent complaints and claims from riparians, and the determined opposition of those who navigated the river, resulted in the abandonment of the idea of the fixed dam and the acceptance of the movable dam in lieu thereof, in spite of its enormous first cost and large cost of operation. Thanks to the long experience of the French in successfully operating movable dams and the valuable experience gained in building and operating several similar dams of earlier date along the Ohio River, it was not a difficult matter to design movable dams of unprecedented height and length. Many interesting features arose during the two years occupied in designing the new system, such as whether transverse rolling gates should be used instead of the more common mitered gates; whether the dams should be built on piling, or excavation be continued to bed-rock at somewhat greater expense; whether the dam foundation should be built solid or be of cellular construction; whether it was desirable to install water turbines in the structure or to take advantage of the low-head power within reach when the dam was up; whether to use the expensive bear-trap weirs, in addition to the long sections of Chanoine weir at each site; and, finally, whether fixed dams of considerable length could not be advantageously used at some of the sites, particularly in the lower river where the fall was slight, the river wide, and ample notice of approaching floods was always available.

It so happened that one of the early dams to be considered was Dam No. 48, below Evansville, Ind. It was evident from a study of the survey of the river that the situation here was different from that several hundred miles farther up stream. Unless the plans were afterward changed for other good reasons, Dam No. 48 contained the innovation of omitting the pair of standard bear-trap weirs. On the Lower Ohio River, there is always sufficient warning of the approach of floods, to afford ample time to lower the Chanoine weir, thus offsetting the great advantage of the bear-trap type of weir, namely, speed and power operation. Furthermore, a decided saving in the cost of the dam and in the time required for construction appeared possible by the omission of the bear-traps. The width of the river at this site was so great that if a standard type of construction, like that in the upper river, had been followed, it would have meant the building of a 700-ft. navigable pass, two standard 91-ft. bear-trap weirs, and 1 400 ft. more of expensive

Chanoine weir. It seemed desirable to reduce the length of weir, and calculations were made with that object in view. The discharge of the Ohio at low-water elevation, at Dam No. 48, was computed to be 7 160 cu. ft. per sec.; at the 7-ft. stage on the navigable pass sill, the discharge became 26 000 cu. ft. per sec.; and at the 9½-ft. stage above the navigable pass sill, which corresponded to a 7-ft. stage above low water, the discharge became 48 000 cu. ft. per sec. As is well known, the discharge for certain stages is greater on a rising than on a falling river. Careful inquiry was made among the lockmasters in the Wheeling District and in the Pittsburgh District to ascertain at what maximum stage of the river, the typical Chanoine wickets of the pass could be raised with certainty. It became apparent that there would be no detriment to navigation, if, on a falling river, the stage was allowed to fall somewhat below 9 ft., as all the larger barges (known as "boats") would have passed down before this, and the raising of the dam could be safely delayed until the stage had fallen to 7 ft. above low water. Therefore, it was concluded in the design of Dam No. 48, that:

First.—It was unnecessary to arrange pass and weir so that the pass could be raised on greater stages than about 9½ ft. over the pass sill or 7 ft. above low-water gauge.

Second.—It would be possible to raise the pass wickets against a head of 5 ft. and this would mean that the final closure of the pass at the 7-ft. stage of the falling river would be made with the upper pool about 13½ ft. above the pass sill and the lower pool about 8½ ft. above this same level.

With the weir at Dam No. 48 designed to pass the discharge of a 7-ft. stage of 48 000 cu. ft. per sec., with the upper pool at 13½ ft. above the sill, any delay in closing off the pass could not result seriously, as the difference between pool elevations would not then increase above the allowable 5 ft., and, as a matter of fact, the lower pool under such condition would begin to re-approach the 7-ft. stage and the head to decrease slightly and this again would cause a slight rise in the upper pool until equilibrium was established. By calculation, it was found that when the upper pool was at 13½ ft. above the pass sill and the lower pool at 8½ ft. above the same point and the river was falling, each foot of navigable pass would discharge about 125 cu. ft. per sec., each foot of Chanoine weir would discharge about 80 cu. ft. per sec., and each foot of the bear-trap weirs, if installed, would discharge about 120 cu. ft. per sec., these figures neglecting velocity of approach and end construction. It then became a simple matter to determine the length of Chanoine weir necessary to discharge 48 000 cu. ft. per sec. Neglecting the 2 000 cu. ft. per sec. as the minimum escaping between the pass wickets, it appears that 600 ft. of standard Chanoine weir should suffice to prevent the head against the pass wickets from rising above 5 ft. on finally closing. If it had been desired to introduce two standard bear-trap weirs in this work, they would have provided about 19 600 cu. ft. per sec. in themselves, and the length of Chanoine weir necessary would have become about 355 ft. As, at that date, two bear-trap weirs, with piers and piping, were estimated to cost $180 000 and the equivalent length of Chanoine weir, 245 ft., with two piers,

would cost only $110 000, a saving of about $70 000 seemed possible by omitting the bear-trap type from this design, the dam to be operated exactly as is done with similar movable dams on the Kanawha River, where success has been attained without the installation of the costly bear-trap weirs. Low-water discharge of the Ohio River at Dam No. 48 was calculated to be 7 160 cu. ft. per sec. Taking 600 ft. as the calculated necessary length of Chanoine weir—no bear-trap weirs to be used—it is found that the remainder of the river can be closed off with a fixed dam of a length of 890 ft., and that its crest can be fixed at 2 ft. below normal upper-pool level. This lowered elevation of the crest of this fixed dam section is such that the ordinary summer and fall discharge will suffice to maintain a full upper pool. However, a design to admit of flash-boards was provided for use under exceptional circumstances of extreme low water. It was also assumed that, at low-water discharge, openings between the pass and weir wickets might be closed with 4 by 4-in. timbers, called "needles". However, to be on the side of safety, it was assumed that there would still be a 1-in. strip of leakage space left between each pair of pass and weir wickets. This assumption gives a low-water loss through the pass of 2 100 cu. ft. per sec., and a similar leakage loss through the Chanoine weir of 1 230 cu. ft. per sec., or a total leakage loss of 3 330 cu. ft. per sec. The remainder of the low-water discharge of 3 770 cu. ft. per sec. would hardly suffice to maintain a full upper pool over the spillway of the fixed dam, but when this unusual condition arose, the flash-boards could be used. The gauge records showed that it would not be necessary to use the flash-boards more than about once in three years.

Further calculations were necessary to determine the maximum discharge that could be passed before it would become necessary to drop the pass wickets. It was assumed that the dam would be lowered when the upper pool rose to a point 6 in. above the tops of the pass wickets when standing. This pool level would give a depth of 11.5 ft. above the Chanoine weir sill and 15.9 ft. above the pass sill. Assuming the accepted design to include 600 ft. of standard Chanoine weir, 800 ft. of navigable pass, and 890 ft. of fixed dam, with crest 2 ft. below normal upper pool, calculation showed that the leakage through the pass wickets amounted to 5 000 cu. ft. per sec. and the 6 in. over the pass crest carried 800 cu. ft. per sec., the 600-ft. weir discharged 61 800 cu. ft. per sec., and the 2½ ft. over the fixed dam spillway carried off 11 000 cu. ft. per sec., or a total discharge of 78 600 cu. ft. per sec. passed the site, which corresponded to an open-river stage of more than 9 ft. above low-water level. This being the condition for which the test calculations were being made, the design seemed satisfactory as it would permit ordinary rises—to somewhat above 9 ft.—to be passed without throwing the main part of the dam and losing the pool. This feature becomes important when one considers what it means to dams farther down stream to be suddenly deluged by the great volume of water released, and also one must consider the many tributaries of the Ohio, which frequently have local rain storms and furnish small floods in themselves, which could be taken care of at the dams on the Ohio without losing the upper pool.

Mr. Hall's suggestion of giving to certain railroads, crossing the Ohio River, the exclusive privilege of operating feeder barge lines on the river, might show up well for the railroad tonnage for the particular road involved and might make the river busier, but it would justly arouse strong opposition from the competing railroads, and would introduce a new element in the rate situation under the supervision of the Interstate Commerce Commission, which rate situation already is sufficiently complicated.

Mr. Hall's statement that "Unfortunately, neither the Engineer Department, nor other Government agency, manages or seriously concerns itself with any part of the transportation problem other than the preparation of a sufficient waterway", is open to discussion. Before Congress adopts any river and harbor project, such as the canalization of the Ohio, elaborate preliminary examination and survey reports are made to cover in detail every phase such as "what is desired by local interests", "proposed plans to carry out the improvement", full investigation of "terminal facilities", the "local co-operation" offered, financial or otherwise, full "commercial statistics" of the locality, "local benefits" to be expected, and "general or national benefits" to be expected. These reports by District Engineers pass the Division Engineer and the Board of Engineers for Rivers and Harbors, each of whom makes full analyses of the District Officers' reports, visits the locality, holding necessary public hearings, and, finally, the reports pass the Chief of Engineers, U. S. A., who makes his own review and transmits the data through the Secretary of War to the proper committee in Congress, with definite recommendations based on every phase of the transportation situation in the locality.

Recently, the Engineer Department transmitted to Congress a report of 2 400 pages, fully illustrated, with large detail maps, on "Water Terminal and Transfer Facilities" of the harbors of the United States. This is one of the most comprehensive documents ever published and shows what great importance is attached to the matter of terminals, as distinguished from the waterways leading to them. Similar elaborate investigations have been made of the rail facilities leading to the ports, the tributary territory, the terminal charges, etc.

The Interstate Commerce Commission is fully empowered to regulate interstate rates by water as well as by rail, and it frequently refuses to grant requests for reductions in rates, as well as increases. Here, seems to lie the agency for protection against "cut-throat" competition.

There is no doubt but that the Federal Government could force a greater use of its rivers by the grant of exclusive privileges, seizure or confiscation of certain rail lines, or arbitrary changes in freight rates, but it is hardly to be supposed that the situation warrants such action. There are many good reasons why, in the United States, commerce should be permitted to seek its own routes irrespective of whether the Federal Government has a substantial investment in waterways, some of which may not be making a satisfactory tonnage showing. What would the situation be if the waterways had been

improved at the expense of private capital, which always demands interest and return of the original investment? This problem looms large to the rail carriers and their security holders.

C. I. GRIMM,* ASSOC. M. AM. SOC. C. E. (by letter).—The author has discussed to a greater or less extent all the engineering features that pertain to the construction of locks and dams on the Ohio River, and particularly to Dam No. 18. This discussion, therefore, will be confined to elaboration on a few subjects which have been of interest to the writer in studying locations and preparing designs and estimates for a considerable number of locks and dams and which are covered rather briefly by the author. These subjects are: (*a*) Foundations; (*b*) weirs; and (*c*) lock-gates.

(*a*).—*Foundations.*—For the first thirty-one dams, it has generally been practicable to build on rock foundation, but below Dam No. 31, such foundations are exceptional. Where rock is not available, dams are supported on wooden piles, generally 30 ft. long, and the present practice is to place a row of interlocking steel sheet-piles, 40 ft. long, under the up-stream edge of the foundation. For the earlier dams, these sheet-piles are of wood, 25 ft. long, and of tongue-and-groove construction. When built on piles, the foundations are protected below the dam by a rock-filled timber crib and an apron of heavy rip-rap.

Although it might appear that for these low lifts, not exceeding 9 ft., the importance of a rock foundation is not significant, this is not true on account of the scour below the weir, which is so extensive that the stability of dams built on pile foundations is endangered to some extent, thus making it necessary to take soundings periodically and place additional rip-rap from time to time. This is especially true in the lower stretch of the river, where the underlying material is sand or contains a large percentage of sand. Where rock foundations have been found, the rock has generally been at such an elevation that the dams were constructed economically by excavating inside an open box coffer-dam, such as was used for Dam No. 18, and placing the concrete foundations on a cleared rock surface. Such construction has been carried to various depths up to a maximum of about 20 ft. below low water. If rock is at a depth of 20 ft. or more below low water, a pile foundation will cost less than one on rock. The advantages of building on rock, however, warrant some additional cost. It has not usually been necessary to give this matter careful consideration, because where the depth of rock was not less than 20 ft., it was so much in excess of this that the cost of building on it was readily seen to be excessive. In the case of Dam No. 32, which is now being designed, the depth of rock is about 23 ft. below low water, and plans for building this dam on rock are being considered. It is proposed to utilize a box coffer-dam of the usual type and inside of this coffer-dam to sink open concrete caissons from the river bed to rock, a distance of about 12 ft. These open caissons, which are comparatively small units, will then be pumped out one at a time and filled with concrete. It is estimated that this construction will cost little more than a pile foundation. At Dam No. 23, where the under-

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lying rock was about 15 ft. below low water, the Dravo Contracting Company constructed the dam (840 ft. long from the lock to the bear-trap) inside of coffer-dams about 45 ft. wide and 160 ft. long, the walls being of a single row of Lackawanna steel piling braced across the 45-ft. width. This work was accomplished speedily and economically.

Ohio River dams constructed on sand and gravel, with a cut-off wall of 40-ft. interlocking steel piles, have a length of "creep" equal to about ten times the maximum head, and where 25-ft. wooden piles are used, it is about eight times the maximum head. Experience has shown that in practice the wooden sheet-piles, even though carefully driven, are not sufficiently tight to form an effective cut-off. Their use has been limited to the upper part of the river where foundations consist of rather coarse gravel and sand, and they are not regarded as adequate for the more sandy foundations of the lower river. No evidence of "boils" under any dams that are in service has appeared, and it is believed that these dams have adequate protection from this danger, unless they are undermined by scouring action below the weirs, which action must be carefully guarded against by proper construction and periodical inspection.

(*b*).—*Weirs.*—The weirs for Dam No. 18 and the earlier Ohio River dams were designed to pass a flow equal to the open-river flow at a stage of from 9 to 10 ft. The apparent advantage of having a large weir capacity is that it lessens the extent of maneuvering the wickets of the navigable pass, but this is an advantage only in case the weir can be maneuvered with greater ease or greater speed than the navigable pass. However, in practice, no more suitable means of raising the wickets of the weir has developed than the maneuver boat. Discharges equal to a 12-ft. stage are passed through the weir and a part of the lowered dam, since the dams are partly raised at stages greater than 9 ft. and are not entirely lowered when the capacity of the weir is reached, unless it is evident that the approaching rise will be high enough to submerge the pass wickets the tops of which are generally about 13 ft. above low water. In practice, a part of the navigable pass, therefore, is used as a weir and the capacity of the weir including the lowered part of the navigable pass is equal to an open-river stage of about 12 ft. As stated by the author, the operation of the Chanoine weir from the service bridge has generally been discontinued, and on account of their inaccessibility to the maneuver boat which operates the movable dam, these weirs are not used. Many of the dams now in operation have, therefore, a usable weir of two 91-ft. bear-traps only. At Dam No. 29, for example, the capacity of the bear-traps is about equal to a 7-ft. open-river stage. This dam is maneuvered successfully to pass any stage up to 12 ft., by lowering a part of the navigable pass. It is necessary, of course, to raise the pass wickets against greater heads than would prevail if the Chanoine weir was in use, and it is rather difficult to raise the last wickets of the pass. In view of the foregoing, it is not regarded as necessary to provide, in addition to the navigable pass, a weir capacity sufficient to discharge a 9-ft. open-river flow and as the cost of the dam is decreased by decreasing the length of the weir, this factor should be given proper weight in the design of weirs.

The bear-trap is generally recognized by those who have had experience on the Ohio River, as the most satisfactory regulating weir that has been devised for this stream, by reason of the fact that it may be operated with greater ease, greater safety, greater speed, and more certainty than any other type. However, in determining the extent of its use, consideration must be given to its first cost, which is high in comparison with that of wicket weirs. Two 91-ft. bear-traps with their piers occupy 220 lin. ft. of dam and will cost more than double the same length of wicket weir. The cost per foot of a weir of Chanoine or Bebout wickets is considerably more than that of a fixed dam, and where a part of this weir may be replaced by a fixed dam, by using some of the navigable pass as a regulating weir, a saving in cost will be effected. Economic considerations, therefore, should limit the weir lengths to those required by the following conditions:

(1) There must be a sufficient area of movable dam and weir so that, when they are lowered, the velocity in the contracted channel will not interfere with navigation.

(2) The weir area must be sufficient so that the pass wickets need not be maneuvered against a head greater than 5 ft.

In the recent designs for Dams Nos. 34, 36, and 38, the weirs have been given considerable attention, and the completed plans have some features which differ from Dam No. 18, or any other Ohio River dams. In these designs, two 91-ft. bear-traps, with the sill 15 ft. below the upper pool, are provided and also a weir, 100 ft. long, of Chanoine wickets, with the sill 11 ft. below the upper pool. The Chanoine weir is provided with a tripping bar for lowering, and this weir is placed between the bear-trap and the navigable pass so that it may be maneuvered by the boat which maneuvers the navigable pass, as the weir wickets will be raised by the maneuver boat. The tripping device is similar to that which has been in successful operation on weirs of the Big Sandy River and the mechanism for operating the tripping bar is on the bear-trap pier.

The weirs for these dams, exclusive of the navigable pass, will discharge a quantity about equal to an 8-ft. open-river stage when pools are at normal height above the dams. The height of the upper pool above low water is as follows: Dam No. 34, 13 ft.; Dam No. 36, 13.7 ft.; and Dam No. 38, 11.6 ft. As the discharge through the weirs will raise the water below to an 8-ft. stage, the maximum heads on the pass wickets, when the weirs are open and the upper pools normal, will be as follows, without allowance for leakage between the pass wickets: Dam No. 34, 5 ft.; Dam No. 36, 5.7 ft.; and Dam No. 38, 3.6 ft. The leakage between the pass wickets will further reduce this head, as will also the raising of the dams below. Furthermore, the raising of the pass wickets is usually completed before the upper pool has filled, although this does not apply when raising only a part of the dam that has been lowered to pass a discharge slightly greater than the weir can pass. In view of these considerations, it appears that the pass wickets, which are used for pool regulation at these dams, will usually be maneuvered against heads of less than 5 ft. and that a weir capacity equal to an 8-ft. open-river

discharge is ample for these dams. The space between the bear-trap and the abutment is closed, by a fixed concrete dam having a minimum length of 252 ft. at Dam No. 36. This places the weirs far enough from the bank that no appreciable scouring of the bank is anticipated. On account of the severe scouring action that occurs below the bear-traps, they should be placed some distance from the river bank if the width of the river at the dam permits it.

(*c*).—*Lock-Gates.*—The lock-gates of Dam No. 18 and of all other Ohio River locks constructed prior to 1916, are of the rolling type. These gates move from recesses back of the land wall across the lock chamber, closing the ends of the locks and forming a water-tight bulkhead. The gate at the upper end of the lock is 17.4 ft. high and that at the lower end 22 ft. high, or somewhat less, depending on the lift. Each gate is mounted on ten pairs of standard car wheels which roll on a steel track.

This type of gate, although somewhat unusual, was selected on account of the limited height of the gates and the great width of the lock, which are far from ideal conditions for the more usual type of mitering gates. The rolling gates, although they have occasioned some minor difficulty from broken wheels and tracks, in general, have operated satisfactorily in the upper river. In the lower river, however, where deposits of mud and sand are extensive during high water, the recesses, unless closely sealed, become so filled with deposits that the cleaning of them involves considerable expense as well as possible interruption to navigation before the lock can be placed in operation. In view of the difficulty and uncertainty of sealing the recesses, the rolling gates were not regarded as satisfactory for the lower river and, in 1914, the Ohio River Board of Engineer Officers adopted a design for mitering gates. As previously stated, the great width of the lock and the limited height of the gates are conditions which are not favorable for mitering gates, but by the use of vertically framed gates, well braced, and with strong bearings and anchorages, a satisfactory design has been effected. These gates have been installed at ten locks and, except for a few minor defects in details, which were readily corrected, have operated satisfactorily, and have been found readily adjustable for any wear, deflection, or warping that has occurred. The recesses for the rolling gates are readily coffered by means of a timber bulkhead and pumped out when repairs or painting is necessary, whereas, for the mitering gates, it is either necessary to install a considerable coffer-dam and pump out the lock, or to lift the gates out by derrick boats. It appears that, in the case of the mitering gates, the latter method and the carrying of spare gates will become necessary if navigation becomes so extensive that it cannot be interrupted except for short periods.

Considerable study has been given to the comparison of first cost of Ohio River locks with rolling or mitering gates, and it has been found that, in general, neither has any marked advantage in this respect when operating machinery and power plant are considered. If the lock is located at a steep rocky bank, the rolling gates are at a disadvantage as it is either necessary to make costly excavation for the recesses or to move the lock out from the bank and this increases the cost on account of the filling and paving required back of the land wall. Except for special cases, the first cost is not a con-

sideration and the advantages of operation, making repairs, replacements, etc., although somewhat balanced for present conditions, would be in favor of mitering gates, if navigation becomes as extensive as is contemplated and hoped for when the improvement of the river is completed.

B. F. THOMAS,* M. AM. SOC. C. E. (by letter).—In recent study of the design of dams on the Ohio River, in the vicinity of Cincinnati, Ohio, considerable discussion has arisen as to several points on which no great emphasis (at least in print) has heretofore been placed. Some of these points will be enumerated by the writer in this discussion. The dams have generally been provided with two bear-trap weirs, a pier separating the first trap (counting from the lock) from the navigation pass. The pass, which is closed by Chanoine wickets, is usually 700 ft. wide, and the width of each bear-trap is 91 ft. Following the bear-traps, there has been provided, at a number of dams, a weir of Chanoine wickets with a sill considerably higher than that of the pass. In recent discussions, it has been suggested that the Chanoine weir be placed next to the pass and that there be no pier to mark the change in sill elevation. It has also been proposed to lower the weir wickets with a tripping bar similar to the method used on Big Sandy River, and on the twenty-seven dams of the Belgian Meuse, the maneuver boat to be used only for raising this section of the dam, but to be used both in raising and lowering the pass wickets, as at present.

The writer would recommend for dams on this part of the Ohio River, the following arrangement: Chanoine wicket pass, 700 ft. long; pier, 12 ft. wide; two bear-traps, each 91 ft. wide; pier, 12 ft. wide between traps; and fixed weir, 181 ft. long, or sufficient to reach the shore abutment.

In case greater weir capacity is desirable, or necessary, he would recommend that a Chanoine wicket weir with tripping bar be placed next the pass, with the pier between it and the pass, the length of the fixed weir being reduced accordingly. Although he does not consider a pier at the end of the pass an absolute necessity, in case a Chanoine weir with a sill considerably higher than that of pass is put in, still it is a desirable arrangement, and it is believed that there is practical unanimity among navigators and river men in general that such a guide for their movement at dams should be provided. The currents are uncertain at the dams and a tow is liable to be carried against the weir and wrecked. The use of a buoy or especially constructed wicket for defining the point of change in elevation of the sill, it is believed, would be unsatisfactory; in fact, it is not believed that a buoy could be maintained in periods of drift and ice.

It is believed that with a sill about 4 ft. higher than that of the pass, little, if any, trouble from gravel interfering with the tripping bar, would be experienced. However, its mechanism and the bar itself should be designed with sufficient strength and power to crush such obstruction to the movement of the bar as might interfere therewith. The fact that at the time the bar is in use, the water is surging at great velocity through the spaces made by the falling wickets would insure, it is thought, the cleaning of the deposit from around it.

* Mr. Thomas, who was U. S. Asst. Engr. at Huntington, W. Va., died on April 14, 1923.

From experience at one dam in the upper river, which has no bear-trap, and from similar experience at dams having only one bear-trap, as reported by the engineer in charge, it would seem unwise to build a new dam with only one trap. The regulation of pool levels where navigation exists is a most important feature in the Ohio River System, and it has been proven that no other device thus far tried or proposed will equal the bear-trap as a regulator. With it, regulation is certain, operation easy and speedy, action reliable, and its work is generally satisfactory; its upkeep is not expensive, and will become less as its working becomes better understood by the engineers in charge.

After much study the writer has come to the conclusion that it would be a serious mistake to substitute any kind of weir heretofore proposed for the bear-trap. The following memoranda are submitted for consideration. As to a pier between a pass and a bear-trap or between a pass and a weir of any type:

(*a*).—The junction of the two sills at different elevations would without a pier form an obstruction to navigation, which cannot be seen by day nor lighted at night.

(*b*).—It furnishes a convenient and firm mooring for the maneuver boat and its lines, during maneuvers, and if its up-stream end is placed flush with the up-stream face of the wickets, it would offer no impediment to the movement of the boat.

(*c*).—It is a guide to navigation; without it, the cross-currents may cause a tow to swing over against it at stages when the weir sill would have insufficient depth for tow.

(*d*).—Navigation has always had this sort of guide to the passage of the dam and would, it is believed, complain greatly and feel uncertain without it.

(*e*).—The experience of the engineers in the Wheeling District indicates that where there are two traps, the maneuver boat was sent out for regulating purposes only five times in a season, while with one trap it went out thirty-six times, and with no trap eighty-nine times. This seems a most convincing argument in favor of two bear-traps. With the large number of tributaries coming in below the dams in the Wheeling District, the writer believes it would be very unwise to put in a dam with less than two traps, especially as the quantity of drift and number of quick, unannounced freshets are great.

MALCOLM ELLIOTT,* M. AM. SOC. C. E. (by letter).—Although the time has not yet arrived for a definite decision as to whether the improvements of the Ohio River and their large cost have been justified to the taxpayers, it would seem to be proper, in discussing this paper, to survey the accomplishments to date, the status of water transportation on the Ohio River, the cost of the work, and the prospects for the future.

Thirty-six of the Ohio River dams are now ready for operation. The series is complete from Nos. 1 to 26, inclusive, Dam No. 26 being just below Point Pleasant, W. Va. (Fig. 1). Dam No. 27 is about 80% completed and will be

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ready for operation during the season of 1923, making the series complete from Nos. 1 to 29, inclusive. Below Dam No. 29, Dams Nos. 31, 33, 35, 37, 39, 41, 43, and 48 are in operation, thus making it possible to fill every alternate pool as far as Dam No. 43, and greatly improving the condition over the open channel as it existed before the construction of the dams. All the even numbered dams intervening, to include Dam No. 44, are under construction, and their completion may be expected within a reasonable time, thus furnishing slack-water navigation from Pittsburgh, Pa., to Leavenworth, Ind., a distance of about 650 miles—the upper two-thirds of the length of the river. Dam No. 48 is the only one below Leavenworth ready for operation.

Below Leavenworth, the river conditions become less favorable to the construction of locks and dams. The bottom is less stable than in the upper reaches, and it is feared that much trouble will be had with deposits of sand, which will make difficult the operation of the dams and lock-gates. The abandonment of the slack-water system in favor of open-channel regulation by dredging and training works in the lower part of the river, has been considered, but it is not known whether any decision has been reached. A little experience at Dam No. 48, near Evansville, Ind., recently completed, will furnish information as to the feasibility of constructing more locks and dams in that section of the river.

The author gives the tonnage for 1894, 1916, and 1917 only. To supplement these figures, it may be said that the greatest tonnage in any year was 14 054 000 in 1900, the smallest, since 1892, was 4 599 000 in 1917, and the tonnage for 1920, the last year reported, was 9 382 000, hauled an average distance of 126 miles, making 1 184 000 000 ton-miles. It will be noted that the greatest tonnage was carried before the slack-water improvements were even begun and that now, with these improvements more than two-thirds completed, it is less than it was without the dams. Therefore, the question might well be asked, would it not have been better to have made no expenditures? This question and some of the factors which have influenced commerce on the Ohio River will be discussed subsequently. First, however, an estimate is given of the costs of the work to June 30, 1921, the latest date for which published figures are available.

The following figures are compiled from the report of the Chief of Engineers, U. S. Army, for the fiscal year ended June 30, 1921:

Total amount expended on new work on all projects to June 30, 1921, including the purchase of the Louisville and Portland Canal, improvements of Pittsburgh Harbor, snagging, and open-channel work	$85 746 412
Total amount expended on maintenance, operation, and care to June 30, 1921	13 268 705
Total of all money spent on Ohio River to June 30, 1921	$99 015 117
Expended for operating and care of locks and dams for year 1920-21	$1 061 973
Maintenance of dredging and snagging, year 1920-21	291 921
Total channel maintenance, operating, and care expense for year 1920-21	$1 353 894

The figures quoted represent only cash disbursements and include no allowance for interest or depreciation, except in so far as the depreciation has resulted in expenditures for renewals. Interest and depreciation should certainly be considered with the operating costs incurred by the Government and by the carrier in computing a financial summary of the operations of any waterway. Because the resulting balance sheet compares unfavorably with a similar statement for a railroad of like mileage, it does not necessarily follow that the waterway is a commercial failure. Its existence might be sufficiently beneficial to adjacent industries, development of territory, and increase of land values, to justify the enterprise, notwithstanding that it is supported in part by Government subsidies.

There is such variation in the length of service of the various parts of a movable dam that computing the proper allowance for depreciation to provide a sum for replacement would be difficult. Some of the wicket irons, for instance, have lasted only 5 or 6 years, whereas the concrete should last 100 years or more. As a sinking fund of $1 per year, if invested at 4%, will produce $100 in about 40 years, it is believed that an allowance of 1% of the total investment in new work, added to the actual expenditures for repairs and replacements included in the annual disbursements, would be a sufficient allowance for depreciation. This is equivalent to the assumption that with yearly repairs and renewals at the present rate, the structures will last for 40 years and that the project should be charged with a sufficient sinking fund to produce the money needed for their replacement at the end of that period. The interest on the capital invested may fairly be assumed at 4% of the expenditures for new work.

With these assumptions, the total annual charges may be estimated roughly, as follows:

Total capital invested to date, about		$100 000 000
New capital needed to finish project		20 000 000
Total eventual cost of project		$120 000 000
Interest and replacement at 5% per year		$6 000 000
Operating expenses, 1921 (thirty-five dams)	$1 062 000	
Operating expenses after project has been completed (fifty dams)	$1 062 000 ÷ 0.7 =	1 500 000
Channel maintenance, dredging, etc		300 000
Total yearly maintenance, operating and care charges, interest, and depreciation		$7 800 000

These figures, although approximate, are probably more accurate than any estimate that can be made of the volume of traffic that will be moved through the agency of these expenditures. If the benefits are to be computed by comparing the present traffic with that prior to the construction of the improvements, then such benefits are nil, because the traffic now is less than before. It is known that such a pessimistic view of the value of the improvements is not justified, because, as will be shown later, events which have no relation whatever

to the river and which could hardly have been foreseen when the project was adopted, have resulted in a large decrease in the river traffic; and without the present facilities the decrease in traffic would have been much more than it has actually been. The benefit to date, therefore, is not measured by the increase in total traffic since the improvements were installed, but by the increase in traffic over what it would have been without the improvements.

At the time the Ohio River improvements now in progress were planned, there existed a large down-stream movement of coal—more than 1 000 000 tons annually—from Pittsburgh, Pa., to Cairo, Ill., and several million tons from Pittsburgh to Louisville, Ky., and intermediate cities. At that time, there was no reason to suppose that this traffic would not continue and, in fact, increase considerably, if channel conditions were improved. This traffic from Pittsburgh to lower river points, however, has almost disappeared. During 1920, for instance, only nineteen coal barges passed Lock No. 13, near Wheeling, W. Va. The abrupt termination of the coal traffic from Pittsburgh was not due to the cost of moving the coal by water, which is, and always has been, less than by rail. Reliable information obtained by the writer, from cost records of a company engaged in transporting coal by water during 1921, indicates the cost to the carrier for a 250-mile haul to be not more than 5 mills per ton-mile, including the short haul by rail from the mine to the tipple, loading, towing loaded barges down stream and empties up stream, overhead, interest, and depreciation. Railroad rates for coal during 1920 were from 7.1 to 14.6 mills per ton-mile for a similar distance. Therefore, the disappearance of the commerce in Pittsburgh coal is not due to the ability of the railroads to haul it at less cost than by water. The general tendency for industries to use coal procured from local sources, thus avoiding the cost of long hauls, is to be recognized in considering this decrease in traffic. A particular instance, having direct bearing on this traffic in Pittsburgh coal, is the greatly increased need for coal in the Pittsburgh District, following its large industrial growth in recent years, especially during the World War, resulting in the local use of a part, at least, of the surplus which in former years went down the river in barges. It is possible, also, that new markets, remote from the river, to which the coal is hauled by rail, have been developed in recent years.

It is difficult to estimate how much traffic has disappeared from the river from the causes named, but an examination of detailed records of tonnage prior to 1912, shows that the average commerce was 10 334 000 tons per year and that with the Pittsburgh coal traffic eliminated, there remained about 6 500 000 tons. The commerce for 1920 was 9 382 000 tons, an increase of nearly 50% in the traffic other than Pittsburgh coal, part of which increase was made possible by the expenditure for improvements. The statistics for 1920 show that there was a traffic of 417 129 000 ton-miles through the locks. If this traffic had been compelled to pay tolls equalling the yearly cost of maintenance, depreciation, and interest charges, about $5 600 000 in 1921, the tolls would have amounted to 13.4 mills per ton-mile. Adding this to the carrier's costs of, say, 5 mills per ton-mile, the total cost of moving the freight would have been about 18.4 mills per ton-mile, as against the prevailing railroad rates for a similar distance (126 miles), of from 12.5 to 13.9 mills per ton-mile.

Such a comparison, however, is not fair to the waterway. One should stop to consider that the project is incomplete; that the outlet by water to the markets beyond the Ohio River is no better than it was before the present project was commenced; that the railroads have built paralleling lines and are allowed to establish lower rates to meet water competition; that there is practically no co-operation between rail and water routes; that railroads charge more for hauling inland the freight from the river than that from other railroads; and that until these disadvantages are overcome, the capital for suitable floating plant, terminal, and transfer facilities cannot be found, and the water transportation companies must labor under the disadvantage of using improper equipment.

Railroads provide branches and sidings so as to connect the source of production with the warehouse of the jobber or consumer, but waterways cannot do this and thus cannot reach their full usefulness except by co-operation with the railroads. This co-operation should include mutual arrangements for shipping goods by rail and water combined, the acceptance by both rail and water carriers of goods for shipment over both routes on through bills of lading, equitable distribution of the through freight revenue between the rail and water carriers, and the provision of convenient terminal and transfer facilities and appliances for transshipment of the freight. There is little, if any, such co-operation on the Ohio River. In its place, there are artificial barriers which must be overcome before the river will amount to anything.

According to the report of the Chief of Engineers for 1921 (Part 2, page 22), railroad rates for similar distances are from 10 to 20% lower when there is water competition than elsewhere. The discrimination against river freight is shown also by the following example: The rate on coal in carloads from Charleston, W. Va., to Indianapolis, Ind., is $2.80 per ton, *via* the Chesapeake and Ohio Railroad, to Cincinnati, Ohio, thence, *via* the "Big Four" to Indianapolis, of which amount the "Big Four" receives $0.91 per ton for its share of the haul, whereas coal brought from Charleston to Cincinnati by boat, then loaded in cars by the shipper, and turned over to the "Big Four", is charged $1.15 per ton for the same haul from Cincinnati to Indianapolis.

Thus, the railroads discriminate against river coal and cause the water carrier to overcome an artificial disadvantage, in this instance, of $0.24 per ton. The effect of such discrimination is to limit the market for river freight to the communities on the river banks.

As long as these artificial and discriminatory barriers exist, the gaining of profits by water carriers is so uncertain that the navigation interests will continue to suffer the disadvantages due to the use of antiquated and improper equipment, because of difficulty in raising capital, and the shippers, being compelled to pay the extra cost of this inefficient service, will be deprived of the benefits which otherwise would be obtained from the waterway.

The Ohio River is favorably situated for commerce. At its upper end, it terminates in the remarkable industrial area and region of mineral resources centering around Pittsburgh. At its lower end, it terminates in the Mississippi River which affords dependable, all-year navigation, either to the Gulf or

to St. Louis, Mo., the former a distributing point for export business, the latter a gateway to the Middle West. Intermediate points along the Ohio River are such cities as Wheeling, producing coal, oil, and steel manufactures; Cincinnati, a manufacturing city and distributing point for other important manufacturing towns; and Louisville, a jobbing center for Kentucky, Tennessee, Alabama, and other States in the South. The Ohio River is fed below Pittsburgh by the Kanawha River, now shipping more than 1 000 000 tons of coal per year and capable of much larger production; the Muskingum River with its head of navigation at the important manufacturing city of Zanesville, Ohio; the Green River, penetrating to the center of the oil and coal-producing section of Kentucky; and the Cumberland and Tennessee Rivers communicating with Nashville and Chattanooga, Tenn. Reaching these important regions and cities, there can be little doubt that the Ohio River and its tributaries are situated so as to carry a large volume of traffic, if only the navigable depths are provided and artificial restrictions removed.

An instance of what can be done with water transportation on inland rivers, is seen in the operations of the Mississippi-Warrior Service. This service has arrangements with the railroads, whereby through shipments, part rail and part water, can be made on through bills of lading. It has modern boats and barges and is constructing efficient terminal and transfer facilities. After operating more than two years at a deficit, it began to earn profits on its Mississippi business, as the tonnage increased during the last three months of the fiscal year ending June 30th, 1921. In this period and the two months following, although the fleets operated at only part capacity and were served by unfinished terminals, the service earned a profit of about $98 000 after meeting all operating expenses and depreciation charges based on the high prices paid for the equipment during and shortly after the World War. The freight rates were from 5½ to 40 cents per 100 lb. lower than all-rail rates for similar distances.

The Ohio River can be made navigable at less cost than the Mississippi; it is situated equally favorably for prospective business; and it, also, can be made into a profitable enterprise if a dependable channel is provided throughout its length and the existing discriminations are removed.

As the author states, an agency is required, the duty of which will be to manage the water-transportation problem other than the preparation of the waterway, but, in his statement that no such agency has been created, it is believed that he has overlooked the Inland and Coastwise Waterways Service, established by Act of Congress during the war, in order to supplement the greatly overloaded railroad systems. This agency is now functioning, and through one of its departments has built up the important Mississippi-Warrior Service mentioned previously. With proper support and sufficient appropriations, this agency should be in a position to foster the growth of the river-transportation business and promote the use of waterways for which many millions have been spent in channel improvements, but it can do little in promoting Ohio River commerce until the lower reaches have been made navigable so as to afford connection with the Mississippi River.

FREDERICK B. DUIS,* M. AM. SOC. C. E. (by letter).—For some years, the writer has been connected with the construction and operation of Ohio River locks and dams in the U. S. Engineer District, Wheeling, W. Va., and, since 1915, Lock and Dam No. 18 to which the author's paper especially relates, has been included in the works under his charge. This discussion is based largely on information gained through experience in maintaining and operating these works.

Attention is invited to some apparent errors in the last paragraph of the historical data under the heading, "Introduction". The locks and dams to Dam No. 39 are stated to have been constructed, but Dams Nos. 25, 27, 30, 32, 34, 36, 38, and 39, at the probable time of the preparation of the paper, were not completed, although Dams Nos. 25 and 39 were nearly so and were expected to be ready for operation at the commencement of the low-water season of 1922. Dams Nos. 41, 43, 44, 45, and 48 are stated to have been placed under construction, but Dams Nos. 41, 43, and 48 were completed and placed in operation in 1921, Dam No. 48, late in the year. At the end of 1921, there were thirty-four Ohio River locks and dams in operation.

Table 2 gives the commerce carried on the Ohio River during 1894, 1916, and 1917. With the statement following this table, those figures indicate (although perhaps not so intended) a gradual decline of the commerce on the stream, in spite of the large expenditure for the construction of locks and dams. The facts are that the decrease in the commerce did not commence in 1894, nor has it continued to date. Reference to the reports of the Chief of Engineers, U. S. Army, will show that from 1894 to 1905 there was an average, although not a regular, increase, the commerce in 1905 amounting to more than 13 000 000 tons, of which about 3 700 000 tons were of coal from Pittsburgh to lower river points. From 1905, an average, but not a regular, decline in the traffic continued until 1917. By 1917, the coal traffic from the Pittsburgh territory to lower river points had practically ceased, the cause of which was in no way related to the river. The commerce previously cited for 1905 included that carried by ferries which, although not separately shown in the published report for that year, amounted to about 2 000 000 tons, as estimated from the amount of such traffic shown in later years when it was reported separately. The commerce given by the author for 1917 does not include the ferry traffic, so proper comparison with previous years when it was included requires that this class of traffic be omitted. Allowing for the 3 700 000 tons of coal carried in 1905, which traffic had ceased in 1917 for causes entirely foreign to the river, and also making allowance for the 2 000 000 tons estimated to have been carried by ferries in 1905 and not included in the author's figures for 1917, the actual decline in the commerce from 1905 to 1917, for which the cause may be attributed to the river, was approximately from 7 300 000 to 4 600 000 tons. Such were the conditions to 1917, but they did not so continue and, in the writer's opinion, the author has erred in not showing the commerce for the subsequent years as available from published reports, as the year 1917 was the turning point in the amount of commerce on the Ohio River and later data give a different aspect on the situation.

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In the published reports available, the commerce for the three years since 1917 is given as follows: 1918, 6 171 412 tons; 1919, 5 400 377 tons; and 1920, 9 382 463 tons. This shows an increase of nearly 100% from 1917 to 1920. The decrease in 1919 compared with 1918 was due to the temporary depression in the industrial activities during the early part of that year. The large increase in 1920 was influenced in the opposite direction by the industrial situation, but the amount of commerce carried in 1920 could not have been possible without the slack-water navigation then available. The question may be asked, why did the turning point in the amount of commerce occur about 1917, when the improvements had been under way for many years and a considerable number of the locks and dams had been in operation for some time? The answer is that not until about 1917 had the improvements progressed far enough to make them of practical benefit to navigation. In 1917, continuous slack-water navigation was made available from Pittsburgh to Dam No. 20, down stream 200 miles, and additional locks and dams providing intermittent pools were in operation from Dam No. 20 to Cincinnati. This practically insured a continuous navigable stage for craft of lighter draft, such as packet-boats, between Pittsburgh and Cincinnati. Craft requiring a greater depth of water than packet-boats could also operate over this section of the river with only infrequent suspensions on account of the river stages. It is, therefore, not a coincidence that, after 1917, the packet business which had almost disappeared from the upper part of the river, began to revive, as the result of the improvements. The foregoing data are in contradiction of the author's statement, in the paragraph on "Commerce", that he thinks the building of locks and dams is not rapidly increasing the tonnage and that under present methods of operation no company carrying passengers and freight in less than barge loads has built up a trade and held it at a fair and profitable rate. The carrying of freight in less than barge loads has revived to a considerable extent and, to the best of the writer's knowledge, at a profitable rate. If it had not, the parties engaged in this business would not be gradually expanding their facilities, as they have been doing, even through the recent period of the most severe industrial depression that has been experienced in the Upper Ohio Valley for many years.

A gradual increase in barge traffic, also, may be expected as industrial conditions improve, in fact, except for coal, it has been increasing in spite of the recent unfavorable conditions. One company, recently organized, has engaged in general towing in barge-load lots, considerable of its business consisting of the transferring of steel products from the Upper Ohio to Mississippi River points. The traffic manager of this company informed the writer that inquries for the services of the company were far more than its capacity to handle with its available plant, consisting of three towboats and a considerable number of barges. One steel company, in the Pittsburgh District, has also commenced transporting some of its products down the Ohio River with its own fleet.

The Ohio River Dam, selected by the author for description in his paper, was finished in 1910, and was the seventh to be completed, of the thirty-four that are now ready to operate. There are a number of items in its design and

arrangements that have been found unsatisfactory and are now obsolete, and the later dams have a number of improvements devised since Dam No. 18 was built. It seems proper to refer to these changes and show the reasons for them, in order that the latest practice in the design of such structures will be on record.

A movable dam is one the purpose of which is to pass high waters without appreciable increase of stage over natural stages, to allow open-river navigation when the open river has sufficient depth, and to increase the depth to a navigable stage during periods of low flow. To answer these purposes, the Ohio River dams have been provided with three principal elements: (1) the lock; (2) the navigable pass; and (3) the regulating weir.

Probably the most complicated of the operations on a movable dam are those connected with the regulation of the pools. The regulating weir is designed to pass water not needed for locking during periods of surplus, in such a way as to maintain a navigable depth in the pool above the dam. Its discharge capacity must be made variable by means of movable parts so as to correspond with the varying flow of the river. There are many conditions to meet, their relative importance is not seen the same by all designers, and the appliances have not always worked in the manner the designers thought they would. For these reasons, there has been a continual change in the design and layout of the regulating weirs since the Ohio River improvements began and, even now, the opinions of the designing engineers have not crystallized into a definite standard arrangement for this important element of a movable dam.

The author's Fig. 8 shows a photographic view of the completed Dam No. 18. The navigable pass is of the same design as that which has been in use on the Ohio for many years, and no further mention of it is necessary, except with reference to the statement made by the author that the pine timber used in the construction of the wickets in this dam is the best wood for the purpose. As observed from service, oak is so much superior to pine in resistance to decay and physical injury that the latter wood is not being used at the present time for such purpose. To the best of the writer's knowledge, the wickets at Dam No. 18 were the first on the Ohio River to be constructed of pine timber, and, a few years later, pine was used at Dams Nos. 20 and 26, but at all the other dams constructed before and since, on this river, oak timber has been used for that purpose, and has demonstrated its superiority.

Considerable difficulty is being experienced in regulating the discharge through Dam No. 18, on account of the inadequate capacity of the two small bear-traps and unsatisfactory operation of the Chanoine wicket weir. The total discharge area of the two bear-traps is 1 350 sq. ft., whereas at all the dams, except one, in 150 miles of river up stream, the two bear-traps provide a total discharge area of 2 730 sq. ft., or more than double that at Dam No. 18. The result is that both the bear-traps at Dam No. 18 can pass less than one-half the quantity of water that can be passed by one of the two bear-traps in the dams above, the combined capacity of which is about sufficient for a 9-ft. stage in the open river, the maximum stage on which it is necessary to operate the dams.

All additional weir area necessary at Dam No. 18 must be provided either by lowering the wickets in the Chanoine weir, for which it was designed, but is little used, or by lowering the wickets in the navigable pass, which was not originally intended, but which is now the usual practice. The amount of weir regulation necessary in this dam is further increased on account of its location between the mouths of two large tributary streams, the Muskingum River, 8 miles above, and the Little Kanawha River, about 4 miles below. These streams are subject to quick rises, with consequent sudden increases in the volumes of water poured into the pools above and below Dam No. 18, making quick action frequently necessary to provide for the sudden change in river conditions. The small discharge area of the bear-traps, therefore, makes frequent additional discharge area necessary for pool regulation. For this, the Chanoine wicket weir was designed, but it is an unsatisfactory and inefficient device. Like the weirs of this type used in nearly all Ohio River dams until a few years ago, it has been a failure and is little used. The unsatisfactory operating results of this weir are in part due to the elevation at which the wickets are hinged. At all other dams where such weirs have been used, the hinge points of the wickets are sufficiently above the center of resultant water pressure to prevent the wickets going on swing. At Dam No. 18, however, as described by the author, the hinge points were located so as to let the wickets swing automatically when the pools rise, in order to increase the discharge opening necessary in the dam. This feature has not been a success. The wickets will frequently swing when not necessary for pool regulation, to prevent which it is necessary to chain them to the service bridge. The idea of automatic swinging of Chanoine wickets for pool regulation cannot be successfully applied on Ohio River dams. The rotation of wickets on their hinges is influenced by both the upper and lower pools, whereas, it should be dependent only on the stage in the upper pool. As there is no uniformity in the differences of elevation between the pools at varying stages, it is impossible to locate the hinge points on the wicket so that they will trip at a predetermined stage of upper pool.

Another factor making it impracticable to use this method of pool regulation is that when the wickets are on swing, the steel frames and bars, known, respectively, as the horses and props, by which the wickets are supported, hold so much drift above them that not only is the discharge capacity considerably reduced, but it is practically impossible to release the drift and right the wickets to close the opening, without lowering the whole dam to reduce the head, and, then, only with an expenditure of much time and labor. Therefore, when such wickets are operated, they must be completely lowered, but this is difficult and slow, particularly with the special winch provided. This winch, although designed for the purpose, is another feature contributing to the impractibility of operating the wicket weir. As stated by the author, too much time and labor are required. The winch is driven by a gasoline engine and is neither suitable in type nor has sufficient power. Even if a steam engine of sufficient power was used, the wickets could not be handled satisfactorily with it, on the heads necessary. The most difficult part of such an operation is the righting of the wickets after they have gone on swing, the nature of

which operation is shown by the author's Fig. 10. The difficulty of this operation cannot be appreciated by any one unfamiliar with it. Operation of the weir with the maneuver boat is a practical impossibility, because one bear-trap is between the weir and the lock side of the river, thus making it necessary to take the boat across the head of the open bear-trap, a difficult and dangerous operation.

These difficulties have resulted in the practical abandonment of the Chanoine weir as a regulating device at Dam No. 18 and other Ohio River dams. Regulation that cannot be done with the bear-trap, is now done with pass wickets, with the maneuver boat. This is possible for such heads as exist after the bear-traps have been opened and the difference in pool levels has been reduced. If the Chanoine weir was between the navigable pass and the first bear-trap, instead of between the two bear-traps, the service bridge could be abandoned and the shorter weir wickets operated with the maneuver boat. This method of pool regulation, however, is much slower and requires considerably more labor than regulating with bear-traps. The wickets also deteriorate rapidly under such hard service and make frequent repairs necessary. For these reasons, wicket weirs are not a satisfactory and efficient substitute for the successful bear-trap weirs developed and standardized on the Ohio River.

The preceding remarks concerning wickets relate entirely to the old Chanoine type. The author describes a new type, the Bebout wicket. Some of these wickets have been used within the last few years, for pool-regulating weirs, as a substitute for one of the two large bear-traps that previously were included in the standard design of Ohio River dams. As described in the paper, these wickets will trip automatically, but they must be raised with a maneuver boat the same as Chanoine wickets. The author cites twenty-four advantages in the use of these wickets. Several years' experience with them has convinced the writer that not all of the claimed advantages can be realized in practice. On account of their automatic tripping feature, these wickets may be used with much benefit and success in some works, especially for weirs in dams where there is not sufficient back-water to effect their tripping, but neither these wickets, nor those of the older Chanoine type, can be used satisfactorily as a main regulating weir in Ohio River dams. The Bebout wickets might be used with benefit for a part of the navigable pass, as will be subsequently explained.

The Bebout wickets are hinged near the middle, like the Chanoine type, but unlike the latter, they lower themselves completely when the resultant pressure is above the hinge point, leaving an unobstructed opening in the dam, thus increasing the discharge capacity and avoiding the holding of drift as in the case of Chanoine wickets, which do not lower automatically, but only go on swing. Leaving an unobstructed opening in the dam is of great advantage, but the automatic lowering feature of the Bebout wicket, just as the automatic swinging of the Chanoine wicket, although theoretically advantageous for regulating weirs in practice, is a detriment, because the automatic tripping cannot be controlled, as previously explained in connection with the swinging of the Chanoine weir wickets at Dam No. 18, and in most cases occurs at other than the theoretical tripping stages.

All the Bebout wickets so far installed, except in one case, have been substituted for bear-traps, replacing one bear-trap of the two previously installed in each dam. In one dam, both the bear-traps were replaced by the wickets. This has not resulted as favorably as anticipated by those responsible for the change and only recently has been abandoned and the two bear-traps have been re-adopted in the standard design for Ohio River dams. The wickets trip too frequently when not necessary for pool regulation or more of them trip than are required. They must be raised with the maneuver boat, the time for which cannot be chosen, but may occur any hour day or night. This requires considerable time and a crew of men that is not available from the regular lock-operating force, except during the day shift. When such maneuvers must be made during the remainder of the twenty-four hours, some of the employees on the regular day shift must be called. This is unsatisfactory and, furthermore, the maneuvers cannot always be made quickly enough to avoid objectionable irregularities in the pool stages. Also, the time consumed by the employees in such duties is usually at the expense of other important activities. In the Wheeling U. S. Engineer District, which includes the dam described by the author, there are dams with two bear-traps, dams with one bear-trap, and one dam without any bear-trap. The force of men employed is the same for the dams with one bear-trap as with two bear-traps, but the loss of morale and increasing complaints as to the amount of work at those dams which have only one trap, and the frequent calls for duty on the dam at all hours of the day and night, make it appear that, in a short time, additional force will be required. At the dam where no bear-trap is used, and all regulating is done with wickets, it is now the practice to employ four extra men during the operating season of about nine months. Therefore, instead of the substitution of the Bebout wickets for a bear-trap requiring a less number of men for operating the locks and dams, as claimed by the author, the facts, as shown by experience, are that the labor is much increased and that it is not feasible to operate such a dam with a smaller force, but that probably such operation will require a larger force in the future. Also, the saving in cost effected by substituting wickets for bear-traps, cited by the author as another of the advantages, is true only for the cost of construction. The cost of maintenance and operation is increased, the amount of which is not known, but probably is at least enough and perhaps more than necessary to compensate for the saving in first cost. The result, then, is no gain financially and a much less efficient and satisfactory method of pool regulation. It is the considerations just cited, which have convinced the writer that, in a long series of dams, the most satisfactory and the most economical appliance for regulating weirs is the bear-trap, as developed on the Ohio River. Enough of them should be used so that when more than their full capacity is required, except in rare cases, it will be time to throw down the dam and use the open river.

The Bebout wickets could be used with benefit for a part of the navigable pass at the end adjacent to the first bear-trap and hinged so high as to trip automatically only when the upper pool has risen to such a depth over the top of the wickets, that the whole dam should be put down. These conditions would usually occur only when the river reached a stage where control of the

dam had been lost and the dam could not be lowered in the regular way by the maneuver boat. The automatic lowering of the wickets would then open a part of the dam and probably lower the upper pool sufficiently to make it possible to lower the remainder of the dam with the maneuver boat. The wickets installed in this way in a dam would afford a considerable protection against having the dam caught up on a stage of river when the lock is submerged and out of commission and navigation cannot pass over the dam. This accident has occurred a number of times, and any protection against it is worthy of consideration.

It seems proper in this discussion to refer to the problem of operating a long series of movable dams, such as that being built in the Ohio River. The problem is to raise the dams on a falling river as it approaches the minimum open-river navigable stage in such a way as to hold the water in the series of pools with a depth sufficient to maintain navigation. If the dams are raised too soon, no particular harm is done except that in case of an immediate increase in the natural flow instead of the expected decrease it may be necessary to lower the dam again and thus the labor of an unnecessary operation will have been incurred. If, however, the raising is delayed too long, the quantity of water in the river channel may be insufficient to fill the pools, and it may take several days for the natural flow of the river to supply the shortage. If one or more of the upper dams of the series have this trouble, they put on needles to close the interstices between wickets and, sometimes, even caulk the spaces with weeds, ashes, etc., so as to hasten the filling of their pools. This immediately results in still further decrease of flow into the pools above the dams in the lower section of the series, aggravating their trouble. Until construction of the Lower Ohio River system of dams is completed, there are and will be stretches of open river through which considerable navigation is attempted. Owing to the great cost of dams and the expected troubles in construction and operation, it may be that the original intention to provide a series of dams all the way to the mouth of the Ohio River will be abandoned, and that navigation in the lower end of the river will be provided by open-channel regulation. In this case, an abrupt stoppage of the natural flow due to filling of the pools in the upper section of the river will cause sudden decreases in navigable depth in the lower part, which will be highly objectionable to navigation.

The ideal solution would be to raise all dams at or before the moment when the water contained in the channel is just sufficient to fill the pools. The operation of a movable dam at any given moment, however, is a practical impossibility. The maneuver boat must be moved to its place, each wicket must be raised separately, and accidents and delays occur. In general, it takes a whole day to raise a dam and unless the labor charge of operation is to be immediately doubled or trebled, it is impracticable to keep a crew available at all hours of the day and night. The river might be falling at the rate of about 1 ft. per day and at the close of the day's work the stage might be, say, 1 ft. above the ordinary raising stage, in which event the lockmaster would expect to raise the dam beginning the next morning. Late in the afternoon, however, several lockmasters farther up the river might have had to raise their dams because of the falling of the river to the closing stage. This produces a "cut"

in the flow, and the lockmaster further down the river finds his stage falling so fast that he cannot get out on the dam and raise it in time to catch his pool. When he gets his dam up, he has to put on the needles and then the men farther down the river have even worse trouble.

With dependable communication by telephone and telegraph between dams, these troubles can be and are very often avoided, but the existing communication is not infallible. Some of the dams are on country telephone lines having from one to a dozen parties, because no other commercial service is available, and frequently all efforts to keep in touch with operations at the other dams fail. A private line down the Ohio River would cost more than $500 000 and entail an annual expense for upkeep, which would hardly be justified by the present commerce. Recent inquiries have been made as to the possibility of using a radio telephone system of communication, and it may be that some such system will be satisfactory. Reliable liaison between the dams is an essential feature of a series such as that on the Ohio River.

W. H. McAlpine,* M. Am. Soc. C. E. (by letter).—The writer has read this paper with much interest, especially the conditions to be fulfilled in the design of the weir and the computations thereon.

The author's calculations "to determine whether the weir will regulate the upper pool when the water is at a 10-ft. stage" (Condition 1), by the formula, $Q = m\ (L'\ H')\ \sqrt{2\ g\ (h + Z)}$, could be made clearer by a statement of the relations of the zero of gauge to elevation of weir sill and pool level or crest of dam. It is not essential (Condition 1) that the weir have sufficient capacity to pass all the discharge at any special stages such as a 10-ft. stage, as part of the pass wickets can be thrown for that purpose. It is believed that the author has taken great liberty in assuming that values of m, derived from observations where the "swell head" was about 0.35 ft., are applicable to a case where the "swell head" is assumed as 1.7 ft. It seems questionable whether this formula should be used for cases of "swell head" as high as 1.7 ft. It is believed that a submerged weir formula is more applicable to the case. Using Francis' formula, with values of n deduced by Herschel, we have:

$$Q = 3.33 b\ (n\ H)^{\frac{3}{2}}$$

$$b = 500;\ n = 0.63;\ H = 11.837$$

Solving, Q = discharge = 33 900 cu. ft. per sec., instead of 57 700 cu. ft. per sec., as found by the author. The formula, $Q = m\ (L\ H)\ \sqrt{2\ g\ (h + Z)}$, appears more applicable to Condition 2, and its use is better illustrated in the paper† by the late Mr. Thomas, referred to by the author. The lack of authentic experimental data as to the proper values of m makes the use of this formula unsatisfactory. It is believed that systematic observations should be made at movable dams to secure more reliable and complete data as to the proper values of the coefficient, m, under various conditions.

* U. S. Asst. Engr., Louisville, Ky.

† *Transactions*, Am. Soc. C. E., Vol. XXXIX (1898), p. 471.

For investigating Condition 3, the author uses the formula of Chanoine and De Lagrene, namely,

$$Z = m\, v^2 \left(\frac{S^2}{S_1^2} - 1\right) \frac{1}{2\,g}$$

in which,

Z = "swell head";
v = mean velocity (before construction of works);
S = discharge area of river before construction of works;
S_1 = discharge area of river after construction of works;
g = 32.2.

Z is assumed as 0.25 ft. and m as 1.05, instead of 1.5, when the "swell head" is reduced from 0.5 to 0.25 ft. After substituting the other known quantities, the value of S_1 = discharge area after construction of works, is found to be 18 250 sq. ft. It is not understood why the value of Z is assumed and the value of S_1 calculated, as the dimensions of the weir and the pass have already been determined, at least tentatively, to satisfy Conditions 1 and 2. It would appear preferable to substitute the tentatively found area, S_1, and calculate the value of Z to determine whether it falls within reasonable limits. No authority is indicated for using $m = 1.05$ instead of 1.5, the value of the coefficient used by the late Mr. Thomas in the paper referred to previously, for cases when the lock-gates are open. The writer has never found any justification for the use of 1.5 for the value of m in the formula, when applied to a lock in a large river, for a case when the lock-gates are open or for any other condition. The writer has compared the value of the "swell head", as determined by this formula, in a number of cases on large rivers with the actual "swell head" and has found that the value of the coefficient, m, should be only slightly in excess of unity. Probably the value of 1.05 given by the author is about correct for the case in question. It is possible that in a narrow cross-section of river, the contractions due to lock walls, piers, etc., might be sufficient to justify a coefficient of 1.5, but certainly not in a wide river, with a dam composed largely of weir and pass sills.

Having found or assigned a value for Z, the author calculates the mean velocity for critical high stage, which is assumed to be at the top of the lock wall, by the formula:

$$V_1 = \sqrt{2\,g\,(Z + h)}$$

in which,

V_1 = mean velocity after construction of dam;
Z = "swell head";
h = velocity of approach prior to construction of dam.

The use of the two previous formulas appears to furnish a rather unsatisfactory and roundabout method of determining the velocity of the section. The values for "swell head" are rather meaningless to the average individual, unless converted into terms of velocity. Unless one is familiar with the subject, one has little idea whether a value of 0.5 or 1.0 is excessive. However, one knows at once whether a velocity of 5.7 or 8.0 ft. per sec. would be excessive. Also, the value found for "swell head" in the formula does not

represent the entire head due to velocity of the current at the dam. To find the total velocity head, the calculated head due to velocity of approach prior to the construction of the dam must be added to the "swell head". In other words, if it is desired to compare the actual measured head due to velocity at the dam with the results from the formula for "swell head", the calculated head due to velocity of approach prior to the construction of the dam must be added to the value found for "swell head". Although the two formulas, $Z = m\,v^2 \left(\frac{S^2}{S_1^2} - 1\right) \frac{1}{2\,g}$, and $V_1 = \sqrt{2\,g\,(Z + h)}$, may be used to determine either the mean velocity after construction of the works (V_1), with an assumed discharge area of dam (S_1), or *vice versa*, the general formula, $V_1 = \frac{Q}{S_1}$ may be used for the same purpose.

Let

V_1 = mean velocity at the dam after construction;
Q = discharge, in cubic feet per second, as determined by the rating curve;
S_1 = discharge area, in square feet, of proposed dam; and
S = discharge area before the construction of the dam.

As $V_1\,S_1 = V\,S$, $V_1 = \frac{V\,S}{S_1}$; but $Q = V\,S$; therefore, $V_1 = \frac{Q}{S_1}$.

If a coefficient other than unity must be applied to allow for sudden contractions, it may be applied to the general formula, $V_1 = \frac{Q}{S_1}$, as well as the formula used by the author. Both the formulas, $Q = m\,(L\,H)\,\sqrt{2\,g\,(h + Z)}$, and $Z = m\,V^2 \left(\frac{S^2}{S_1^2} - 1\right) \frac{1}{2\,g}$, are modifications of the general formula, $Q = A\,V$, and their value depends on the completeness and reliability of the observations made to determine proper values of the coefficients to be applied under various conditions.

Some of the areas of the cross-section of the river bed given in Table 3, for various stages, are apparently in error. For example, the difference in area between the 10-ft. and the 11-ft. stages is given as (20 540 — 18 350) 2 190 sq. ft.; and between the 11-ft. and the 12-ft. stages, there is a difference of only 1 270 sq. ft. Again, S is given as 24 600 sq. ft. for a 16.4-ft. stage, or an increment of only about 600 ft. for each additional foot, assuming the area of 21 810 sq. ft. for a 12-ft. stage to be correct.

The principal object of this discussion is to bring out the desirability of more complete observations as to actual velocities created by the construction of locks and dams and other obstructions, in order that reliable coefficients may be available for the formula selected for use.

GARDNER S. WILLIAMS,* M. AM. SOC. C. E. (by letter).—Several years ago, the writer was confronted with the problem of providing, in a storage dam, a gate

* Cons. Engr., Ann Arbor, Mich.

through which large quantities of logs could be sluiced at variations of pond level through a range of 20 ft. The width of the opening required by the lumbering interests was 30 ft. After considerable investigation, his attention was directed to the three-leaf bear-trap gate of the so-called Lang type, built in 1894, on the Chippewa River at Chippewa Falls, Wis. It had a length of 80 ft., with a rise of 6 ft., and had then been in service more than 20 years. An examination of the design of this structure, which was of timber, indicated that by the use of steel the type could be readily applied to a rise of three times that in the existing example. A similar timber structure came to notice later. It was built in 1891 on the St. Croix River at Nevers, Wis., and had a crest length of 80 ft. and a rise of 16 ft.

The so-called Island Lake Dam of the Great Northern Power Company on the Cloquet River, about 22 miles northeast of Duluth, Minn., was thereupon equipped with a bear-trap gate of the Lang type, having a rise of 20 ft. and a span of 30 ft. As far as the writer is aware, this is the highest rise under which a bear-trap gate has thus far been operated. The gate was placed in service in 1915 and has been in satisfactory use since.

The success of this gate has led to the use of this type by the writer for the discharge of flood waters in connection with power developments on the Huron River in Michigan. At the Geddes Plant of the Detroit Edison Company, there are two such gates, each having a span of 24 ft. and a rise of 11 ft. At the Superior Plant of the same Company, the writer has installed a similar gate, the operation of which is made automatic by the use of a siphon which exhausts the water from under the gate when the pond rises above a desired level. The seal is broken when the water surface is lowered, and the gate then rises to its original position.

As the operation of these gates simply requires the opening or closing of an inlet or outlet valve, they can be readily manipulated by one man, and if used in connection with a power plant, motors can be installed on the valves and the apparatus controlled from the power house at any desired distance.

A description of the Island Lake Dam has been published,* as well as a valuable Symposium on Movable Dams† which is interesting not only as to matters of design, but also as to the history of the bear-trap gate. The Sector Dams of the Sanitary District of Chicago at the Lockport Power House have also been described,‡ as well as the 160-ft. bear-trap dam§ above Lockport.

In the gates designed by the writer, it has been found desirable to use rollers under the up-stream end of the idler. Such rollers had not been generally used on the timber structures previously built, but when metal members are in contact, rollers are frequently necessary.

MORRIS KNOWLES,‖ M. AM. SOC. C. E. (by letter).—The author's noteworthy contribution on the design and construction of movable dams on the Ohio River is of timely interest to the student of rivers and of the effects

* *Engineering News-Record*, August 2d, 1917.
† *Journal*, Assoc. of Eng. Societies, June, 1896.
‡ *Engineering News*, November 12th, 1908.
§ *Loc. cit.*, March 28th, 1898.
‖ Cons. Engr., Pittsburgh, Pa.

of man's regulating and restrictive works thereon. It may be worth while to note some of the discussions and opinions as to the effects of movable dams on flood heights and the care necessary to insure sufficient water for lockages during times of low flow.

On page 98, the author states:

"It is well known that a project substituting fixed dams for movable dams would be less expensive; but the requirements of navigation *and the popular fear of an increase in flood heights* made the building of fixed dams in this river appear at that time to be impracticable."*

And, on page 140, Col. Harts states:

"The Ohio is subject to extreme floods. Sometimes, as much as 70 ft. in range is recorded, and great injury and loss invariably result to the towns along the banks. Nothing could be allowed in the bed of this stream that would be likely to increase this range, *and in the public mind fixed dams were objectionable for this reason.*"

The writer remembers the discussion and conferences that took place in 1915 when the project for the substitution of a fixed dam at Emsworth, about six miles below the Point at Pittsburgh, for the movable type of dam at Davis' Island, called Dam No. 1, was under consideration. Many were apprehensive that this new dam, about 21 ft. high above the bed of the stream, would seriously increase the flood heights at Pittsburgh. The writer was asked to review the situation for the City of Pittsburgh and the Pittsburgh Flood Commission. He was much gratified to report, after many studies and calculations, that a disastrous flood of 35.5 ft., such as occurred in 1907, would be increased at the Point by less than 1 ft. by the erection of such a fixed dam, and that an extreme flood, 10 ft. higher, which might happen by a combination of fortuitous circumstances, would be increased to a negligible degree. Thus, when this report became known, much of the opposition to the fixed type of dam at this location disappeared.

Much has been said about the tightness of wickets, gates, bear-traps, etc., and the attendant possibility that, during low-water stages, there may not be sufficient water for the large number of lockages required for full navigation potentialities. The author states:

"Considering the question of tightness, Condition 7, it is believed that bear-trap dams can be made as tight as any other type of dam. Needles, curtains, gates, and, possibly, other types can be made tighter than wickets. With the use of wickets, calculations should show a shortage of water not more than once in 10 to 20 years. * * * Computations indicate that, with pass wickets 15 ft. high, with weir wickets, 12 ft. high, and with the pass and weir of the lengths proposed, the leakage through the interstices between the wickets will about equal the discharge of the river at a 3-ft. stage. Therefore, during several weeks of each year, needles over the interstices will be necessary."

Again, on page 139, Col. Brown states:

"In the operation of this system of locks and dams, much trouble has been experienced by reason of inability to raise the dams at stages higher than about 9 ft. This results in the impounding and holding back of the discharge of the river in the pools of those dams that are raised, with a consequent rapid

* The italics in this and other quotations in this discussion are the writer's.

falling off in the discharge of the river at points lower down. * * * So great has this effect on the discharge of the river become, that at times much delay and obstruction to navigation has been experienced at and below Louisville, by reason of the manipulation of the dams at and above Cincinnati. As the number of dams in operation increase, the effect on the lower part of the river will become more and more marked, and, at times *may result in an almost complete absorption of the discharge of the river* if great care is not exercised in the raising of the upper dams."

That this fear of insufficient lockage on streams made artificially navigable by locks and dams is a real menace, is shown by the following statement from the report of the Pittsburgh Flood Commission, published in 1912, after a long period of careful investigation and research:

"Under present conditions there are serious troubles with shortage of water on the Monongahela River during dry weather. * * * At such times the pools farthest upstream are drawn down to furnish water for the pools nearer Pittsburgh, and through navigation is impossible, the upper pools being sometimes lined with barges waiting for water to float them downstream. * * * The reports of the Chief of Engineers of the United States Army give the minimum discharge of the Monongahela as 160 sec-ft. which occurred in 1895, the dryest season in recent years. This is only about two-thirds of the discharge of 244 sec-ft. necessary to supply water for 180 lockages, even if there were no other losses."

It thus appears there is danger when complete navigation facilities will have been provided and if they are utilized to the utmost of traffic capacity. The report quoted previously states elsewhere that the low water of the Monongahela will be increased to six times and that of the Allegheny to three times its minimum by the use of storage reservoirs. The Ohio at Wheeling, W. Va., 90 miles below the point of particular observation at the time of the study, will be raised 2.3 ft. at the time of lowest stages.

Surely an amplification of such a system is all that is needed further to improve conditions farther down stream. It is a mistake to argue that the beneficial effect of storage is negligible, except just immediately below a reservoir constructed for such a purpose. The effects on flood flows and the raising of low stages are likewise noticeable many miles away. The writer holds no brief for contemplated accomplishment of navigable depths by letting out stored waters as a general and universal proposition, but to say that such have no use in augmentation and helpfulness is an unnecessary attempt at a reversal of the laws of hydraulics.

On page 133, Mr. Thomas P. Roberts has stated,

"It has been proved that to reduce materially the height of great floods at Pittsburgh, restraining reservoirs should be as near as possible to the city, for the flood-producing storms reaching Western Pennsylvania are only rarely more than 90 miles in width of excessive precipitation. * * * Had the 1913 storm moved eastward only 80 miles farther, careful study indicates that, at Pittsburgh, the flood would have been at least 15 ft. higher than the record flood of 1907. It is useless to attempt to estimate the damages which would have resulted from such a flood to the towns, bridges, etc., above Pittsburgh. * * * *To talk of restraining reservoirs for such floods, at least for the area above Pittsburgh,* is waste of time."

The carefully prepared studies in the report of the Pittsburgh Flood Commission, previously referred to, which was approved by a Board of United States

Army Engineers as to practicability and feasibility, in 1913, states the highest known flood at Pittsburgh, that of 1907, would have been reduced 10 ft., or to a stage of 25.3, only 3.0 ft. above danger, by the construction and proper manipulation of seventeen selected reservoirs out of a total of forty-three possible sites. Other floods, which had occurred over a period of 50 years, would have been reduced below the danger line. Surely this controverts such an extravagant statement of the over-zealous objector. It is a good principle of life to secure the good we can, even if one's 100% desire cannot be obtained.

In this connection, it is interesting to note the remarks by Col. Brown on page 139:

"To meet this difficulty, a comprehensive scheme of control of operating the dams will have to be devised, so that constant reports as to the stages of the river and variations of discharge may be received at a central point, at which will then be determined the proper time for the raising and lowering of all dams, thus regulating them so as to affect the discharge in the lower parts as little as possible. A telephone system is being provided, connecting all dams to a central point, which will probably be Cincinnati, and that will be the point from which this centralized control will be exercised. After some means can be devised whereby the dams can be raised at stages above 9 ft., much benefit will be gained, and it is understood that attempts are now being made to devise some method of raising the dams at higher stages."

This is just the sort of managerial control that is exercised in inter-related gravity water supply systems and is exactly what the Pittsburgh Flood Commission planned for the comprehensive system of reservoirs proposed by it.

The author's remarks regarding the Government's activity in water transportation evidently caught the attention of Maj. Adams, who pertinently asks the question in conclusion:

"What would the situation be if the waterways had been improved at the expense of private capital, which always demands interest and return of the original investment? This problem looms large to the rail carriers and their security holders."

Again, as Col. Harts so aptly states on page 142:

"The supreme test of the public value of any inland waterway must always be an economic one. * * * How many of the inland streams, on which extensive navigation projects have been built, can now meet this test?"

It is hoped to have soon a legislative determination, based on an enlightened public opinion, that munificent expenditures for improvements to navigation are only warranted—if judged alone, without other effects as to the economic advantages to transportation to be obtained thereby—when all the fixed charges and overheads, as well as operating and maintenance expenses, to be borne either directly or indirectly by the traffic, are less than for some other means of transportation and distribution of products. When this enlightened day comes, perhaps there will be less opposition to the acknowledgment of the thesis that a stream should be considered as a whole from its source to its mouth. Thereby, many potential uses will be studied and amplified and some of the costs of the work will be distributed over many objects, and the burden to any one will be proportionately lessened.

WILLIAM M. HALL,* M. AM. SOC. C. E. (by letter).—Mr. T. P. Roberts seems to think that an erroneous inference may be drawn from the paper, to the effect that his distinguished father not only recommended movable dams, but recommended the particular types of movable dams finally adopted. Such interpretation was not intended. To be more exact, on page 57 of the report of 1870 by the late Mr. W. Milnor Roberts, reference is made to a "plan of movable hydraulic gates for chutes designed by T. R. Brunot", and there are other references to movable dams in that report. On page 132, a definite recommendation as to the character of the improvement proposed states that:

"from Pittsburgh down * * *, slack water navigation would appear to be the most desirable, but for the reasons which are given in my study of the plan of locks and dams, I think that none but low dams with chutes should ever be placed there."

The writer believes it to be evident that Mr. W. Milnor Roberts fully intended that the chutes should be closed with some type of movable dam. As narrated by Mr. T. P. Roberts, for 30 years or more prior to that report and recommendation, much interest had been manifested in the type of project for the improvement to be adopted, and several pamphlets were printed and distributed describing the proposed improvements and their respective advantages. The definite recommendation by Mr. W. Milnor Roberts is the earliest known to the writer for the design of dams with a movable part for the purpose of reducing the height of the fixed part. At about that time, Addison M. Scott, M. Am. Soc. C. E., under direction of the late Gen. W. P. Craighill, U. S. A., Past-President, Am. Soc. C. E., was commencing the improvement of the Great Kanawha River. He visited Europe, inspected the movable dams of France and Belgium, recommended the use of the Chanoine wicket for the dams of the Great Kanawha improvement, which were adopted by Gen. Craighill and the War Department, and commenced their construction in the early Seventies. A few years later, Col. Merrill adopted the Chanoine wicket for the movable part of Dam No. 1, Ohio River. It is believed that the first Chanoine wickets built and placed by the United States Government were on these works. With that beginning, as stated by Col. Harts, Division Engineer, Central Division (including the entire river), much work and study have been required to plan the project, with all its mass of details as it is now formulated. The writer has read many of the engineering reports on the river, made during the past 70 years, and fully concurs with Col. Harts in regard to the character of the improvement, and that the best, of the many types of construction proposed, have been selected.

The remarks on page 96, in reference to the possible slow permanent growth in traffic, refer especially to package traffic by the public carrier, the "packet-boat" or "packet-line". On the upper river, where the channel and other conditions are favorable, barge-line traffic by privately owned barges and boats is already progressing under favorable commercial conditions, except for the interchange of traffic from barge line to railroad, which is usually believed to

* Parkersburg, W. Va.

be with a discrimination against the barge lines. Therefore, the barge traffic by privately owned barges and boats should expand as the improvement progresses down stream, and it appears to be doing so. The statement by Mr. Roberts of the tonnage capacity of some of the locks is a valuable record. However, as it is believed to include only coal in barges loaded to capacity, it would hardly be wise to use it as basic data for estimating the capacity of the Ohio River locks for passing a mixed tonnage with possibly less that 50% of coal. The important thought which the writer wishes to convey, to those not familiar with the Ohio River improvement, is that it is converting a river with a past navigation of intermittent character and frequent annual or monthly suspensions to one which is continuous, reliable, and of sufficient capacity to admit of any probable growth or expansion which may be desired in case of war, railroad strikes, or the suspension of railroad growth, or in case of other local or National calamity. With this improvement completed and properly maintained, it will be only a question of providing the requisite boats and barges and manning them and possibly of building a second lock in 25 to 50 years. It is believed that it is the part of wisdom for communities, States, and the Government to provide reserve resources in transportation, especially when it can be done at such a small first cost, and by a method which involves a minimum of cost for operation and maintenance. A highway of equal extent, permitting an equally quick expansion in tonnage by automobile, or any kind of railway, held in continual readiness for use, will cost in annual upkeep probably ten times that of the Ohio River locks, dams, and channel. Although the United States has a wonderful railway system, one of the lessons of the World War was that the transportation system was insufficient for such an emergency, hence the great importance of providing the reserve in transportation afforded by the Ohio River.

The description of the industrial conditions in the Ohio Valley, together with the economies of the project by Col. Harts, is a valuable addition to the reference thereto on pages 95 and 96. The opinions relating to river traffic are somewhat divergent, and conclusive reasons for a failure to have a great increase already in the package traffic are so difficult to determine definitely, that it would hardly be advisable for the writer to dwell on them here, but he thinks the main cause for the failure, however, is stated in the paragraphs, near the middle of page 96. The interesting discussion of the subject by Maj. Elliott, with a statement of the traffic to June 30, 1921, and the economics of the project, is elucidating, and may satisfy the suggestion by Col. Harts on page 141. As the annual traffic through some of the locks in the Pittsburgh District has exceeded 25 000 000 tons, it appears reasonable to expect that, by the time the project is completed to Dam No. 48, below Evansville and Henderson, the average annual commerce for the Ohio River will be not less than 12 000 000 tons, with an average movement of at least 200 miles, or 2 400 000 000 ton-miles. Notwithstanding Maj. Elliott's interesting discussion, it appears that, after the effects of the swollen after-war rates of interest and prices are past, the annual interest on first cost, plus the cost of operation, care, and maintenance may be expected to be about $6 000 000. These estimates make the annual fixed charges per ton-mile for the entire commerce about 2½

mills. An examination of "Poor's Manual" does not disclose corresponding figures of average cost per ton-mile for railway track maintenance plus the interest on first cost of the completed permanent way; however, the average revenue per ton-mile for all the railways of the United States is given as 7.4 mills for 1920, 6.46 mills for 1919, 5.99 mills for 1918, 4.56 mills for 1917, and 4.06 mills for 1916; that is, an average of 4.06 mills per ton-mile in 1916, the year of low prices before the war, as compared with the average of 6.62 mills for the years, 1918 to 1920, of swollen prices due to the war. Although the writer has not obtained the corresponding statistics for 1921 and 1922, it is known that the railway revenues have continued to rise, and when these statistics are completed, the average revenue for these two years will probably be nearly 1 cent per ton-mile. As it is believed that the average interest and dividend rate on railroad bonds and stocks does not exceed 5% annually and is probably nearer 4¼ to 4½%, it becomes evident that the railway rate for interest on first cost, which is believed to be about 2 mills, added to the maintenance, must be 2½ mills and probably considerably more, that is, as much as for the tonnage of the average railway of the United States as for the Ohio River, or more. As many of the railways in the Ohio Valley, under present traffic are working to their capacity, and as the 2 400 000 000 ton-miles for the river can easily be multiplied five to ten times, when such quantities of tonnage are offered, it appears that, as a transportation line for the entire valley, the improvement is unquestionably advisable, and much more desirable than additional railway trackage of an equal cost. It is believed that a good double-track railroad for the entire length of the river, ready for operation, would cost 50% more than the completed improved river.

To the public, another advantageous feature of the improvement is that referred to on page 96, "railroad low rates at competitive points", and more strongly emphasized by Maj. Elliott at the bottom of page 1284. It is so well known that water competition lowers railroad rates at all competitive points that it does not seem to be necessary to present more statistics or discussion. It appears that these advantages of emergency insurance in case of National or industrial war, and low freight rates effected by water transportation are not given due weight by many writers and speakers on this subject. The writer is so confident that this river will ultimately comply with the financial test enunciated by Col. Harts, Maj. Adams, and Mr. Knowles, that, in this particular case, these other great reserve advantages need not necessarily be used as governing factors. However, it is not clear to the writer that the test, as enunciated, is the one to be taken as the final mandate by the Government in all cases, because it is well known that the principal benefits of the railways of this country are not to the stockholders and bond holders but to the people who use them, who own property near them, and to the National body politic. The Government is now spending many times as much for highways and National paved roads as for rivers and harbors. No such test as that referred to seems to have been suggested as a condition for that expenditure. Why, then, should such a test apply and be the final criterion for river improvement any more than for public highways for automobiles,

horses, and pedestrians from which no revenue is received or expected except in the way of National prosperity and the tax thereon?

In the movement of package freight, the writer believes that a change in the present system of operation must come before the volume approaches that which should exist and be expected for a proper service to the public. It must be understood that the building up of extensive service of that character for the public requires financial responsibility, an organized and permanent force of trained agents equal to that of the railways, with no possibility of "cut-throat" competition, a continuity and permanency of service equal to that of the best railroads, some uniformity in quality of service and schedules of boats, and an interchange of traffic with the railroads, with one bill of lading only from shipper to consignee, whether or not the delivery is made by railway or by boat. With these defects remedied and such conditions made equal to those of the best railroads, there is a large tonnage of package freight moving by railways along the banks of the river which can be delivered by boats more economically and in less time than by rail, thereby resulting in economy and convenience to the public. Maj. Adams' remarks on this subject indicate that he may have overlooked the writer's suggestion that any award of special privileges should be under proper Government competitive methods and control. The essential accomplishment desired is that the "packet lines" from the head to the mouth of the river establish and maintain a permanent service, for the country people, towns, and cities, equal in accommodation, and reliability, and superior in speed of movement to the best railway service. Until such a service is realized a lack of genius as a people to manage efficiently affairs of that character will continue to be displayed.

The writer appreciates Col. Brown's discussion of the conditions in the lower river as a valuable extension of the brief reference thereto on page 98, especially as to the reasons being considered and the interesting features introduced by the omission of two or more of the dams below Dam No. 49. As he states, the short season of low water, the great fluctuations in stages, the great quantities of shifting sand, and the failure to find rock at any of the sites on which to found masonry makes the estimate of first cost high; and the shifting sand will no doubt make the cost of operation greater than that for those dams above Cincinnati. With the new 9-ft. channels through obstructing bars planned so as to have the direction of flow of the current parallel to the axis of the channel at all stages, the probabilities of success appear to be favorable.

In reading Col. Brown's statement (page 139), of the time required to place rolling gates in service after the winter floods, it should be remembered that, although two weeks' time is required at two or more locks below Cincinnati, the cases are exceptional. The delay in first maneuvers of the gates after floods is usually due to the accumulation of deposits of great depth on the gate tracks and about the gates. In the recesses which are placed so as to have a current flow over them at a rate equal to the average channel flow, the accumulation of deposit during one winter is not serious. The deposit in the recesses of Lock 18, after the three winters, 1910 to 1913, averaged 3 ft. in depth at the portals and 5 ft. at the rear ends; the maximum depth not

exceeding these averages more than 1 ft. This small depth of deposit is due in part to the location of the lock on the concave shore. Where other conditions make it desirable to select a location on a straight stretch, or on a slightly convex shore, in order to obtain the swiftest current possible, the recesses should be as far out in the channel as other conditions will permit. A location directly below a jutting point, on a bar, immediately above a bar, or in a natural eddy is likely to cause excessive deposit. Deposit is not easily remedied by covering the opening. To be effective, the covers must form practically a perfect seal, and as they must be broken as soon as the flood is past, that method is expensive.

As Col. Brown states, in this long series of movable dams there has already occurred a serious shortage of water immediately after raising the dams, resulting in stages of possibly as much as 3 ft. below the natural stage for periods of several days.

The writer has read Mr. Knowles' report to the Pittsburgh Flood Commission, referred to in his discussion. As apparently anticipated in his discussion and stated by Col. Brown, it appears that "a comprehensive scheme of control of operating the dams will have to be devised", or, at least, that appears to be desirable. In devising a complete system of control, it may be well to consider the possibility of using stored water and of obtaining some of the supply from the pools of tributaries with fixed dams now in existence, and also the further possibility of increasing such storage from dams proposed for future construction on tributaries.

The system of steel sheet-pile coffers in use for the lock and abutment at Dam No. 34, to which Col. Brown refers, was started under his direction, for the purpose of reducing the great cost of coffers for work of this class, which, on the middle section of river, is nearly $200 000 for a single lock and dam. It consists of a series of small coffers ranging in size up to 49 by 150 ft., built of a single wall of steel interlocking sheet-piling about 40 ft. long, driven in rectangular or curved forms, and braced by several tiers of 12 by 12-in. timber. This method has been tried to a more limited extent at three up-river dams, and on many smaller industrial works. This is its first adaptation to all the parts of an Ohio River lock with a deep foundation of piles. The principal advantages of the steel coffer appear to be:

(*a*) It is not so likely to be injured by floods, and permits starting coffer construction a month or more earlier than with either of the wood types.
(*b*) Less excavation is required.
(*c*) Less damage to the coffer by floods and by deposit, and less machinery in the coffer to be submerged.
(*d*) Less obstruction in the river, thereby not so much increase in the velocity of the current, or obstruction and interference with navigation.
(*e*) The coffer wall is closer to the permanent work, thereby enabling derrick boats to stand outside the coffers and place loads in the permanent work.

The principal disadvantages are believed to be in the slower progress in seasons of long low water, excess in cost during seasons when there is no damage to wood coffers by floods, and excess in difficulty of removal, especially where piles are driven deep. It is also evident that the cost of the steel coffer construction increases much faster by increase in the span between the steel walls than the distance. This cost becomes excessive when the timber braces are longer than 20 or 24 ft., and require splicing or are purchased in unusually long lengths. A comparison of the cost reports of the building and removal of Lock 34 coffer and of those of Lock 38 box coffer, which was constructed and in use during the same time, indicates that the cost of Dam No. 34 is the greater.

Maj. Adams' data of the hydraulics of the river at Dam No. 48 (Mile 804) and his description of some of the main features of the dam is a valuable addition to the paper. However, when the dam was designed, about 10 years ago, it was hardly realized what a great improvement the improved bear-trap would make in the ease of regulating the upper pools, and the great reduction it effects in the labor of operation. In recommending that one of the Dam No. 18 bear-traps be placed next to the abutment, it was expected that part of the difficulty of passing from pass to wicket weir would thereby be overcome. As indicated on pages 111 and 128, it has long since been clear to the writer that it would have been wise to have placed both traps together and adjacent to the abutment, even at the further expense of increase in the cost of bank protection. It is now gratifying to learn, from the discussions by Mr. Grimm and the late Mr. Thomas, that they also had reached the same conclusion in regard to the relative position of pass, wicket weir, and bear-traps for other dams now being designed or constructed. As the 91-ft. bear-traps can be relied on to maneuver in 1 or 2 min., up or down, without any other effort than that required in opening two valves and closing two others, and, as the up keep is small, it appears to the writer to be inadvisable to build a dam without one trap; two are preferable; and, as stated on page 129 and by Mr. Grimm on page 149, the excessive cost of the bear-traps is one of the principal reasons for not adding a third trap instead of 100 or 200 lin. ft. of weir wickets, as in the dams of recent design. However, another reason, which also must be considered, is the fact that a head of water is required to remove deposit from the trap recesses and inlet and outlet culverts immediately after the wickets are raised. The wider the bear-trap opening the longer the time needed to obtain the required head. At that particular time, in maneuvering the dam, every minute of delay is of importance to navigation. The reader should bear this condition in mind while reading the long argument (pages 159 to 164) for more bear-traps and the entire elimination of weir wickets.

Mr. Williams gives an interesting record of the trap of highest lift of which the writer knows.

In reading Mr. Duis' reference to the Dam No. 18 bear-traps and weir, it should be remembered that the dams on the upper river were not built in order. When Dam 18 was started and when the bear-traps were designed and constructed, there was not a steel bear-trap in operation on the river, or in the world, approaching it in stiffness or weight per square foot of lower leaf. At

that time only Dam No. 1, with only one 50-ft. wooden trap, had been placed in operation. The writer visited that dam in 1902, and the trap was out of service for repairs. It is the great improvement, in the No. 18 and the other steel traps with leaves of the same design, which has reduced the usage of the Chanoine wicket weir, notwithstanding the excellence of weirs of that type operated from a bridge, as long demonstrated by the eight dams on the Great Kanawha River. The six dams referred to on page 160 are Dams Nos. 1 to 6, inclusive, on the Ohio River, all of which were commenced several years in advance of Dam No. 18. Wooden traps were first designed for several of them and for some of the other dams. The change to steel for Dams Nos. 2 to 5, inclusive, was decided at about the time the No. 18 plans were completed or after. The span of the steel traps for those dams was made 91 ft. to fit masonry which had been built before the decision as to building the steel traps, and thus that span has become the standard for the river. When Dam No. 18 was designed, the regulation of all the upper pools then in operation in the United States was being done principally or entirely by the use of wickets. That method had been used extensively in Europe for many years. The operation of wickets from a bridge is the only method of pool regulation on the Great Kanawha. Although the improved bear-trap is superior to the wicket as a regulating weir, under the direction of the builder of the Kanawha dams, their operation appeared to be excellent until the No. 18 type of trap was placed in commission. During the 5 years of operation of Dam No. 18, under the charge of the writer, the wicket weir was used as an important factor in regulation and operation. In later years, due to the ease of maneuvering the bear-traps, the inclination appears to be to use the traps for that purpose to their limit before seeking assistance by use of the wickets. Although the improved bear-trap has reduced the use of the weir wickets, it seems somewhat premature to state that wickets are "obsolete". This seems especially true as long as those in authority continue to design and build new dams with wicket weirs, as indicated in the discussions by Mr. Thomas and Mr. Grimm. The writer regrets that they did not amplify more of the subjects introduced so briefly in the paper.

Besides the improved arrangement of wicket weirs in Dams 34, 36, and 38, described on pages 149 and 151 it is gratifying also to note that, in some of the new plans, the filling and emptying of culverts for the bear-traps is being planned about as suggested in Paragraph (*c*), page 130.

In referring to the formula, $Q = m\ (L'\ H')\ \sqrt{2\,g\,(h + Z)}$, page 103, Mr. McAlpine pertinently says that "the lack of authentic experimental data as to the proper values of m makes the use of this formula unsatisfactory". The writer diligently sought a more applicable formula, and more data as to values of m, without satisfactory results. He doubts if any exist. It is believed that satisfactory data must be obtained from completed dams in operation, and not from models. In the particular case used on page 103, from observations of the discharge through the completed dam, the writer believes that the actual discharge is quite close to that computed (57 700 cu. ft. per sec.), and he is confident that the discharge computed on page 165 (33 900 cu. ft. per

sec.), equivalent to the natural discharge of the river at a 7-ft. stage, is too small. The writer fully agrees with Mr. McAlpine that it is time that some of these formulas and data were tested by experiment, and that all desired values be obtained by careful observation and velocity measurements at all stages below the crest of dams.

The apparent error in the areas of discharge, referred to by him, is due to the fact that the gauge observations were made at a point about ½ mile below the dam, up to a stage of 10.7 ft., the highest stage at which observations were made. These areas, from a section at that place, were used up to the 11-ft. stage, whereas, above 11 ft., the areas were taken from the cross-section of the river on the axis of the dam.

In 1855, Mr. Herman Haupt, a Civil Engineer, published an article on the Ohio River problems, improvement of navigation and flood control by use of restraining dams, in which he says, "the control of floods, therefore, may be left to future legislation. It will be sufficient at present to attend to the improvement of the nagivation. To accomplish this object will be glory enough for one generation."

Most members of this Society were born after these words were written, and have seen engineering inventions and works, greater, more numerous, and more wonderful than those by all the preceding generations combined. They indicate a surprising vision for one of that day when the wonderful discoveries and improvements of this generation were hardly commencing.

It is a great pleasure to the writer to know that he has taken even a small part in the solution of the problems relating to the Ohio River project and improvement, one of the many great works conceived, designed, and executed by his fellow workers of this Society.

As a last word, the writer wishes to express sincere and grateful appreciation to each of the contributors for the valuable matter added to this paper, by their discussions.

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Paper No. 1511

THE COMPARISON OF CONCRETE GROINED ARCHES AS AN AID IN THEIR DESIGN

By Philip O. Macqueen,* Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Thomas H. Wiggin and Philip O. Macqueen.

Synopsis

Most engineers recognize the value of comparison as an aid in the design of practically all structures, both as a valuable check and also as a preventative of unusual errors. Comparison, however, must be used with good judgment and extreme caution, as otherwise dangerous mistakes may easily be made. Comparisons cannot be used, in some instances, with special or original structures, but these cases are a small proportion of the total.

With statically indeterminate structures, such as concrete groined arches, the use of comparison as an aid to design becomes doubly valuable. Actual design in this case is impossible, and the engineer must use empirical methods and comparisons. Empirical formulas alone could not be used safely in the design of concrete groined arches, and comparisons, therefore, become fundamentally important.

This paper presents a study of the proportions of typical concrete groined arches which have been used successfully for reservoir roofs. It is believed that the various ratios and dimensions discussed will be helpful in problems of design and that the use of these ratios will save not only a great deal of the work of preliminary design, but will serve also as a guide to more economical structures. Special types of concrete groined arches built to support unusually heavy loads, such as filter tanks, are not included in the scope of this paper.

As one of the problems of Civil Engineering the groined arch, now generally used for reservoir roofs, is an interesting anomaly. The method of designing

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the structure is empirical and from a strictly mathematical or even logical viewpoint, its exact design is impossible. This type of structure has been used, however, for more than 2 000 years, and, when carefully constructed, its safety is beyond question. It is as safe or, perhaps, safer than the ordinary arch used in bridge or in building construction. The singularity of the groined arch roof lies in the condition that a structure so simple, strong, and harmonious, is so complicated as regards a mathematical analysis of its stresses, even under a symmetrical loading. The purpose of this paper is to make a comparison of existing groined arches and to suggest several rules that should be of use in the design of such structures.

The usual methods attempted in the design of concrete groined arches are based on the principles that are used in the design of simple cylindrical arches. These methods, as applied to concrete groined arches, have been fully developed by Thomas H. Wiggin,* M. Am. Soc. C. E., and also by Leonard Metcalf,† M. Am. Soc. C. E. It is unnecessary, therefore, to repeat the complete mathematical analysis. The method, although simple, is somewhat cumbersome, and is not completely presented in textbooks. The article by Mr. Wiggin contains several methods of designing groined arches, but for the purpose of this paper, it will be sufficient to summarize briefly the generally preferred method.

In general, there are two steps in this preferred method of design: First, the selection of a groined arch of certain dimensions; and, second, the investigation of this arch for stability under the required loadings by the "hypothesis of least crown thrust." This hypothesis assumes that the true "line of resistance" is that for which the thrust at the crown of the arch is the least possible consistent with equilibrium. The strength and stability of the arch depend on the position of the "line of resistance" and the position of this line, in turn, depends on the amount, direction, and point of application of the crown thrust. The crown thrust is assumed to act horizontally at the upper third point of the thickness of the arch at the crown. The external loading in this case is considered as acting on a triangular area obtained by assuming that the arch is cut vertically along the groined lines, as this method of loading will be found to give greater unit stresses at the "joint of rupture" than other methods. As a condition of equilibrium, it is further assumed that the center of pressure will always remain within the middle-third of every joint. With these assumptions and correct data in regard to the dead and the live loads, the crown thrust and the thrust at the joint of rupture are easily (although somewhat laboriously) computed by the usual methods as given in textbooks on simple cylindrical voussoir arches. The final step in the design consists in laying out a cross-section of the arch on a large scale, and making a careful graphical check of the problem and the position of the "lines of resistance" for the various loadings.

The preferred method of design previously outlined, although cumbersome, has always produced safe structures and, therefore, it is used. With good

* *Engineering Record*, March 12th, 1910, p. 298.

† *Transactions*, Am. Soc. C. E., Vol. XLIII (1900), p. 37.

concrete, the factor of safety is high, as has been proved many times. Arches designed for the normal loading of 2 ft. of earth plus a live load of 100 lb. per sq. ft. have safely carried a load of 10 ft. of earth. Cases are also on record of other arches carrying heavy concentrated loads far in excess of those anticipated. As an example of a still more severe test, arches unsupported at the crown have been loaded to at least four times the allowable load, thus proving the strength of the structure. Groined arch roofs have failed, but, in each case, the failure has been due to faulty foundations, poor concrete, or to unusual stresses at certain times during construction. No cases are on record of groined arches that have failed on account of poor design or even excess loading.

TABLE 1.—Data on Concrete Groined Arches for Reservoir Roofs.

No.	Date	Location.	Clear span ($2a$), in feet.	Rise of intrados (b), in feet.	Crown thickness (t), in feet.	Rise of extrados (h), in feet.	Span, center to center, of columns ($2c$), in feet.	Ratios: $\left(\frac{b}{2a}\right)$	Ratios: $\left(\frac{h}{b+t}\right)$	Average thickness of roof, in feet.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	1908	Watertown, N. Y.	10.000	1.500	0.500	0.833	11.500	0.150	0.417	0.592
2	1909	Providence, R. I.	10.250	2.500	0.500	1.250	11.917	0.244	0.417	0.635
3	1903	Ithaca, N. Y.	10.500	1.500	0.500	0.833	12.000	0.143	0.417	0.568
4	1909	Springfield, Mass.	11.333	2.000	0.500	1.250	13.000	0.177	0.500	0.566
5	1911	Toronto, Ont., Canada	11.333	2.000	0.500	1.167	13.000	0.177	0.467	0.582
6	1910	Owen Sound, Ont., Canada	11.500	2.500	0.500	1.167	13.000	0.218	0.388	0.625
7	1903	Washington, D. C.	11.833	2.500	1.500	1.417	13.667	0.211	0.472	0.606
8	1899	Albany, N. Y.	11.917	2.500	0.500	0.500	13.667	0.210	0.167	0.774
9	1900	Superior, Wis.	12.000	2.500	0.500	0.500	13.667	0.208	0.167	0.747
10	1903	Washington, D. C.	12.167	2.500	0.500	1.417	14.000	0.206	0.472	0.600
11	1912	Roland Park, Md.	13.000	2.667	0.500	1.500	14.667	0.205	0.473	0.598
12	1907	Philadelphia, Pa.	13.167	3.000	0.500	1.750	15.000	0.228	0.500	0.609
13	1907	Lawrence, Mass.	13.167	2.750	0.500	1.667	15.000	0.209	0.513	0.590
14	1908	Pittsburgh, Pa.	13.167	3.000	0.500	1.750	15.000	0.228	0.500	0.609
15	1904	Philadelphia, Pa.	13.417	3.000	0.500	1.750	15.250	0.224	0.500	0.607
16	1913	Baltimore, Md.	13.500	3.000	0.500	1.750	15.000	0.222	0.500	0.582
17	1912	Grand Rapids, Mich.	13.833	3.000	0.500	1.833	15.000	0.217	0.524	0.563
18	1907	Philadelphia, Pa.	14.000	3.000	0.500	1.750	15.833	0.214	0.500	0.604
19	1902	Milford, Mass.	14.000	3.000	0.500	1.500	16.000	0.214	0.428	0.658
20	1909	Springfield, Mass.	14.000	2.750	0.500	1.500	16.000	0.196	0.462	0.623
21	1916	Cleveland, Ohio.	14.080	3.500	0.500	2.000	15.750	0.249	0.500	0.618
22	1908	Columbus, Ohio.	15.167	3.167	0.500	1.917	16.833	0.209	0.522	0.566
23	1905	Washington, D. C.	15.500	3.500	0.500	2.083	18.000	0.226	0.522	0.646
24	1908	Pittsburgh, Pa.	15.750	4.000	0.500	2.167	18.000	0.254	0.482	0.683
25	1912	New York City.	16.000	3.750	0.500	2.000	18.000	0.234	0.470	0.653
26	1913	Minneapolis, Minn.	16.333	3.500	0.500	2.000	18.000	0.214	0.500	0.600
27	1913	Montreal, Que., Canada.	17.000	4.250	0.500	2.000	19.000	0.250	0.422	0.700
28	1912	New York City*.	18.000	4.500	0.500	2.500	20.000	0.250	0.500	0.646
29	1912	New York City*.	19.160	5.000	0.500	2.580	22.000	0.261	0.469	0.744
30	1920	Cleveland, Ohio*.	17.960	4.500	0.500	2.500	20.292	0.253	0.500	0.675

* Under construction or proposed.

Notwithstanding this proved strength, however, the method of designing groined arches is empirical, and a strictly mathematical or even logical method of their design is impossible. This statically indeterminate condition is due principally to the multiplicity of the points of support so that, although the loading on the groined arch roofs is symmetrical and easy to compute, it is, nevertheless, practically impossible to develop a formula that will give a logical distribution of the stresses caused by the loading. The flat slab type of concrete structure is statically indeterminate for the same reason. The

preferred method of design for groined arch roofs is that which gives the greatest unit stresses and, therefore, the highest factor of safety. The labor involved in this method can be appreciated by the fact that the average engineer, unless experienced in this work, usually will require a month to lay out, investigate, check, compare, and finally select a groined arch for an important structure. Simple, rational formulas have replaced the lengthy methods originally used in the design of flat slab roofs so that it is consistent with economical engineering practice that, if possible, the complicated method of design of groined arches should be replaced with much simpler methods.

Strength and economy are the two most important items in the design of groined arches. Table 1 presents a comparison of thirty typical groined arches, the dimensions and general data for most of which have been taken from a table compiled by John H. Gregory, M. Am. Soc. C. E., and published in an article* by J. W. Armstrong, M. Am. Soc. C. E. The arches are listed according to the length of span. The ratios given in Columns 9 and 10 and the average thicknesses of the groined arch roofs given in Column 11, have been calculated by the writer.

Table 2 gives a summary of the various unit stresses and crown thrusts obtained in a few typical arches. In each case, the stresses were calculated by the method of "least crown thrust," as outlined previously.

TABLE 2.—Unit Stresses and Crown Thrusts Calculated by Method of "Least Crown Thrust" for Three Typical Groined Arches.

No.	Clear span ($2a$), in feet.	Rise of intrados (b), in feet.	Crown thickness (t), in feet.	Rise of extrados (h), in feet.	Span, center to center, of columns ($2c$), in feet.	Arch Stresses for Loading of 2 Ft. of Earth, Plus Live Load of 100 Lb. per Sq. Ft.			
						Crown thrust, in pounds.	Maximum unit compression at crown, in pounds per square inch.	Maximum unit compression at joint of rupture, in pounds per square inch.	Maximum unit shear, in pounds per square inch.
1	17.96	3.75	0.50	2.25	20.292	5 310	148	267	59
2	17.96	4.00	0.50	2.25	20.292	5 120	142	238	55
3	17.96	4.50	0.50	2.50	20.292	4 640	129	210	51

Tables 1 and 2 present a number of interesting comparisons which will be discussed in the order of their importance.

(*a*).—*Form of Groined Arch Roofs.*—The type of groined arch reservoir roof invariably selected by designing engineers consists of one in which the intrados is formed by semi-elliptical cylindrical surfaces and the extrados by parabolic cylindrical surfaces. The arches listed in Tables 1 and 2 are of this type. The elliptical shape of the intrados lends itself to a systematic arrangement at the column supports and the flattened form of the arch allows for long spans with a minimum loss of head-room. The parabolic extrados,

* *Engineering Record*, November 15th, 1913.

in its turn, gives a uniformly increasing thickness to the arch. Taken together, the two curves are excellently adapted for groined arch reservoir roofs, and it is difficult to conceive that more effective curves could be used.

(*b*).—*Length of Span.*—The length of span is more a question of economy than of strength, although strength is an important item for investigation. If the columns are long, the span should also be long, in order to save concrete by a reduction in the number of columns. The calculations must also include the average thicknesses of the roofs, and several comparisons should be made. Confidence in the strength of groined arch roofs has increased rapidly in the past twenty years, and the tendency is to use longer spans than was customary in the early days. Spans as short as 10 ft. would hardly be considered at the present time, unless the groined arch roof was intended to support very heavy loads, such as filter tanks. For columns less than 20 ft. in length, the average span used at present is about 15 ft., center to center of columns; whereas for columns 20 to 30 ft. in length, the span is usually 18 to 20 ft. A span of 20 ft. is probably the maximum which could be recommended for groined arch roofs.

(*c*).—*Crown Thickness.*—As shown in Table 1, the crown thickness in every case has been taken as 6 in., and the comparison, therefore, is simple. It might be argued that the crown thickness for the arches with the shorter spans could be less than that given, but this is not the case. With the loads in question (normally 2 ft. of earth plus a live load of 100 lb. per sq. ft.), the 6-in. thickness is about the minimum that could be recommended. Engineers feel responsible for the safety of the structures they design and hesitate to use excessively thin sections, although calculations may indicate that such sections are thicker than necessary. It will be readily seen, however, from Table 2, that a thickness greater than 6 in. is uncalled for, as all the compressive unit stresses at the crown are less than 150 lb. per sq. in. A reasonable increase in the loading, or in the length of the span, would possibly give a maximum compressive unit stress at the crown of 300 lb. per sq. in., but even this would be safe. The use of 6 in. for the crown thickness for all concrete groined arch reservoir roofs, therefore, does not seem to be unreasonable.

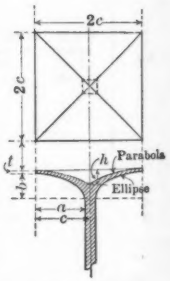


FIG. 1.

(*d*).—*Average Thickness of Roof.*—In order to make a study of the economy of the several groined arches selected for comparison, it is necessary to calculate the volume of concrete contained in each. This study is easily made by formulas derived by Mr. John H. Gregory and W. B. Fuller, M. Am. Soc. C. E. The volume, V, of a single square groined arch roof section (Fig. 1), above the springing line of the intrados, is calculated as follows:

$$V = m + n - p$$

$$m = 4b\left[c^2 + \frac{2}{3}a^2 - \frac{\pi}{2}ac\right]$$

$$n = 4c^2 t$$

$$p = \frac{2}{3}c^2 h$$

in which,

a = one-half the clear span;
b = rise of intrados;
c = one-half of the span, center to center, of columns;
h = rise of extrados; and
t = crown thickness.

When the clear span, crown thickness, and span, center to center, of columns have been selected, the quantities, $4\left[c^2 + \frac{2}{3}a^2 - \frac{\pi}{2}ac\right]$ and $4c^2t$, become constants which may be expressed by k and k'. The formula for volume then becomes:

$$V = b(k) + k' - \frac{2}{3}c^2h$$

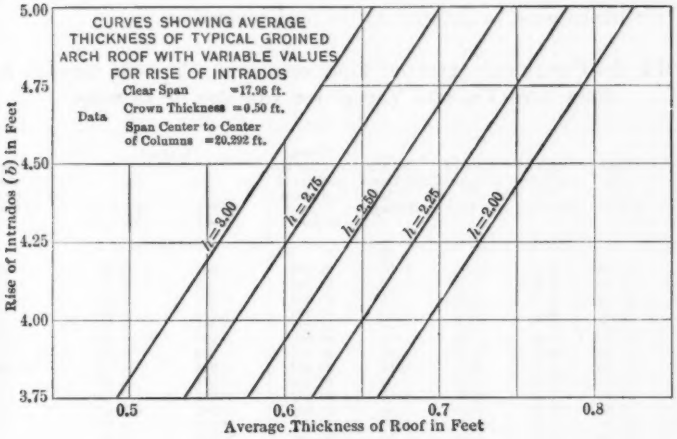


FIG. 2.

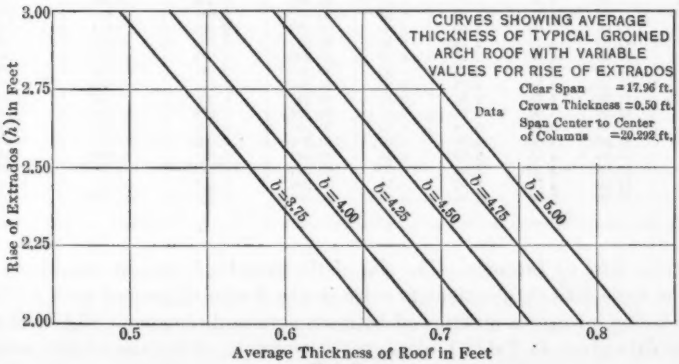


FIG. 3.

The average thickness of the groined arch roof, which is a convenient method of expressing the volume, is then obtained by dividing the volume by $4c^2$, the total projected area of the groined arch section. All dimensions should be given in feet. Tables 3 and 4 have been calculated in order to show the effect of the change of the rise of the intrados and extrados on the volume of concrete. In Table 3, the rise of the intrados has been made the variable and, in Table 4, the rise of the extrados has been made the variable. The curves, Figs. 2 and 3, have been plotted from the results given in Tables 3 and 4. It will be observed that the average thicknesses of the groined arch roofs increase directly with the increase in the rise of the intrados and inversely with the rise of the extrados. For the purpose of comparison, tables and curves similar to those just mentioned should be prepared for each new span selected for investigation. It will also be found that the average thickness decreases directly with the span. The volumes of concrete contained in the columns must be included in the final comparison.

TABLE 3.—CALCULATED AVERAGE THICKNESSES OF A TYPICAL GROINED ARCH ROOF WITH VARIABLE VALUES FOR THE RISE OF INTRADOS.

No.	Clear span ($2a$), in feet.	Rise of intrados (b), in feet.	Rise of extrados (h), in feet.	Crown thickness (t), in feet.	Span, center to center, of columns ($2c$), in feet.	Ratios.		Average thickness of roof, in feet.
						$\left(\frac{b}{2a}\right)$	$\left(\frac{h}{b+t}\right)$	
				0.50				
1	17.96	3.75	2.00	0.50	20.292	0.209	0.471	0.661
2	17.96	4.00	2.00	0.50	20.292	0.223	0.445	0.693
3	17.96	4.25	2.00	0.50	20.292	0.237	0.422	0.726
4	17.96	4.50	2.00	0.50	20.292	0.252	0.400	0.758
5	17.96	4.75	2.00	0.50	20.292	0.265	0.381	0.792
6	17.96	5.00	2.00	0.50	20.292	0.278	0.364	0.825
7	17.96	3.75	2.25	0.50	20.292	0.209	0.530	0.618
8	17.96	4.00	2.25	0.50	20.292	0.223	0.500	0.652
9	17.96	4.25	2.25	0.50	20.292	0.237	0.474	0.684
10	17.96	4.50	2.25	0.50	20.292	0.252	0.450	0.718
11	17.96	4.75	2.25	0.50	20.292	0.265	0.428	0.750
12	17.96	5.00	2.25	0.50	20.292	0.278	0.409	0.782
13	17.96	3.75	2.50	0.50	20.292	0.209	0.588	0.577
14	17.96	4.00	2.50	0.50	20.292	0.223	0.556	0.610
15	17.96	4.25	2.50	0.50	20.292	0.237	0.526	0.642
16	17.96	4.50	2.50	0.50	20.292	0.252	0.500	0.675
17	17.96	4.75	2.50	0.50	20.292	0.265	0.476	0.708
18	17.96	5.00	2.50	0.50	20.292	0.278	0.454	0.742
19	17.96	3.75	2.75	0.50	20.292	0.209	0.647	0.536
20	17.96	4.00	2.75	0.50	20.292	0.223	0.612	0.567
21	17.96	4.25	2.75	0.50	20.292	0.237	0.580	0.602
22	17.96	4.50	2.75	0.50	20.292	0.252	0.550	0.634
23	17.96	4.75	2.75	0.50	20.292	0.265	0.524	0.666
24	17.96	5.00	2.75	0.50	20.292	0.278	0.500	0.700
25	17.96	3.75	3.00	0.50	20.292	0.209	0.706	0.494
26	17.96	4.00	3.00	0.50	20.292	0.223	0.667	0.526
27	17.96	4.25	3.00	0.50	20.292	0.237	0.632	0.558
28	17.96	4.50	3.00	0.50	20.292	0.252	0.600	0.593
29	17.96	4.75	3.00	0.50	20.292	0.265	0.572	0.625
30	17.96	5.00	3.00	0.50	20.292	0.278	0.545	0.657

(*e*).—*Rise of Intrados.*—The rise of the intrados is usually considered one of the more difficult questions to solve in the design of groined arch roofs, as this factor involves a question of both strength and economy. Fig. 4, plotted from data given in Table 1, shows the comparison of the rise of the intrados to the clear span. The rise of the intrados is observed to increase gradually

with the increase of the clear span, and the average rate of this increase is shown by the curve in Fig. 4, the equation of which is:

$$y = \frac{x}{3} - 1.5$$

or,

$$b = \frac{2\,a}{3} - 1.5.$$

TABLE 4.—Calculated Average Thicknesses of a Typical Groined Arch Roof with Variable Values for the Rise of Extrados.

No.	Clear span ($2a$), in feet.	Rise of intrados (b), in feet.	Rise of extrados (h), in feet.	Crown thickness (t), in feet.	Span, center to center, of columns ($2c$), in feet.	Ratios.		Average thickness of roof, in feet.
						$\left(\frac{b}{2a}\right)$	$\left(\frac{h}{b+t}\right)$	
1	17.96	3.75	2.60	0.50	20.292	0.209	0.471	0.661
2	17.96	3.75	2.25	0.50	20.292	0.209	0.580	0.618
3	17.96	3.75	2.50	0.50	20.292	0.209	0.588	0.577
4	17.96	3.75	2.75	0.50	20.292	0.209	0.647	0.535
5	17.96	3.75	3.00	0.50	20.292	0.209	0.706	0.494
6	17.96	4.00	2.00	0.50	20.292	0.223	0.445	0.693
7	17.96	4.00	2.25	0.50	20.292	0.223	0.500	0.652
8	17.96	4.00	2.50	0.50	20.292	0.223	0.556	0.610
9	17.96	4.00	2.75	0.50	20.292	0.223	0.612	0.567
10	17.96	4.00	3.00	0.50	20.292	0.223	0.667	0.526
11	17.96	4.25	2.00	0.50	20.292	0.237	0.422	0.726
12	17.96	4.25	2.25	0.50	20.292	0.237	0.474	0.684
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18	17.96	4.50	2.50	0.50	20.292	0.252	0.500	0.675
19	17.96	4.50	2.75	0.50	20.292	0.252	0.550	0.634
20	17.96	4.50	3.00	0.50	20.292	0.252	0.600	0.593
21	17.96	4.75	2.00	0.50	20.292	0.265	0.381	0.792
22	17.96	4.75	2.25	0.50	20.292	0.265	0.428	0.750
23	17.96	4.75	2.50	0.50	20.292	0.265	0.476	0.708
24	17.96	4.75	2.75	0.50	20.292	0.265	0.524	0.666
25	17.96	4.75	3.00	0.50	20.292	0.265	0.572	0.625
26	17.96	5.00	2.00	0.50	20.292	0.278	0.364	0.825
27	17.96	5.00	2.25	0.50	20.292	0.278	0.409	0.782
28	17.96	5.00	2.50	0.50	20.292	0.278	0.454	0.742
29	17.96	5.00	2.75	0.50	20.292	0.278	0.500	0.700
30	17.96	5.00	3.00	0.50	20.292	0.278	0.545	0.657

The rise of the intrados to be used in the average groined arch roof, therefore, may be expressed as being approximately equal to one-third of the clear span minus the constant, 1.5. The effect of the rise of the intrados on the strength of the groined arch is shown in Table 2, in which it may be noted that the unit stresses and the unit shares decrease steadily with the rise of the intrados. Referring to Tables 3 and 4 and Figs. 2 and 3, however, it is observed that the average thickness of the groined arch roof increases directly with the increase of the rise of the intrados. These two conditions, therefore, are opposed to each other, and it is necessary to make a careful study of both in order to arrive at a logical selection of the most suitable dimensions.

The ratio of the rise of the intrados to the clear span is given in Column 9 of Table 1. As this ratio varies between the approximate limits of 0.20 and 0.25, the rise of the intrados, therefore, may also be expressed as approximately equal to one-fifth of the clear span for the shorter spans and one-fourth of the clear span for the longer spans.

(*f*).—*Rise of Extrados.*—The rise of the extrados used in the average groined arch is given in Table 1, in which also (Column 10) is given the ratio, $\frac{h}{b+t}$, of the rise of the extrados to the sum of the rise of the intrados and the crown thickness. In general, this ratio is observed to have an approximate value of 0.50, and the average rise of extrados to be used, therefore, may be expressed as being equal to one-half of the total rise of the arch, that is, the sum of the intrados and the crown thickness. It will be observed further that the unit stresses increase directly with the rise of the extrados and that the average thickness of the arch increases inversely with the rise of the extrados. These two conditions, therefore, are also opposed to each other, and it is necessary to make a careful study of both in order to make a logical selection of the most suitable dimensions.

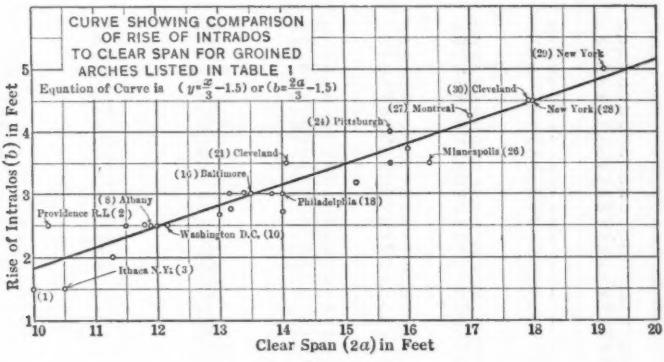


FIG. 4.

(*g*).—*Unit Stresses and Loadings.*—As shown in Table 2, the compressive unit stresses obtained in the calculations for groined arch roofs are much lower than those normally used in the design of other concrete structures. As a result of this condition, the specifications generally call for concrete containing a relatively small quantity of cement, the proportions of the mix usually selected being either 1:2.5:5 or 1:3:5. The saving in cost of the cement alone over a 1:2:4 mix, therefore, is from 5 to 10 per cent. In a structure containing 10 000 cu. yd., and with concrete at $15 per cu. yd., this saving would be about 2% of the total cost. In this connection, it is well to call attention to certain other phases of this question. In the first place, the unit stresses obtained are hardly more than guesses and it is not altogether wise to place too much confidence in the calculations. In the second place, the calculated unit shears are relatively high and there is, perhaps, danger in this condition in case extremely lean concrete mixtures are used. In general, it is believed to be unwise to use concrete mixtures leaner than 1:2.5:5, and, in many cases, a 1:2:4 concrete is perhaps preferable.

Summing up the various comparisons, it will be convenient to express the final results in the form of a few simple rules, as follows:

(*a*).—The form of groined arch roof in which the intrados consists of a semi-elliptical cylindrical surface and the extrados of a parabolic cylindrical surface is excellently adapted to this type of construction and, therefore, it should be used.

(*b*).—Spans from 15 to 20 ft., center to center, of columns may be used with safety. Economy will always call for long columns with long spans. In selecting the most suitable length of span, the calculations should always include the volume of concrete in the roof.

(*c*).—A crown thickness of 6 in. may be used satisfactorily for practically all groined arch reservoir roofs.

(*d*).—The average thickness or volume of the groined arch reservoir roof should be calculated carefully for all trial arches selected, as this factor is an important guide in the determination of the most suitable arch to be used.

(*e*).—In general, the rise of the intrados (taken only to the nearest 0.25 ft.) should be about equal to one-third of the clear span minus 1.5, that is, $b = \frac{2\,a}{3} - 1.5$.

(*f*).—In general, the rise of the extrados (taken only to the nearest 0.25 ft.) should be about equal to one-half of the sum of the rise of the intrados and the clear span, that is, $h = \frac{b + t}{2}$.

(*g*).—The unit stresses obtained in the calculations should be used with caution, and it should be realized that these stresses are somewhat problematical. In general, the proportions of concrete used in groined arch roofs should be not leaner than a 1:2.5:5 mix, and, in many cases, a 1:2:4 mix is preferable. The normal loading for the roof is usually taken as 2 ft. of earth plus a live load of 100 lb. per sq. ft.

The question of whether to use a groined arch or a flat slab roof for a reservoir is one which frequently involves differences of opinion. Both types are acceptable, although, in certain cases, one type is preferable to the other. For small, shallow reservoirs of an area of, say, less than 1 acre, the flat slab roof may be more economical, on account of the lower cost of form work, and, perhaps, better standardized methods of construction. For larger and deeper reservoirs, however, covering an area of several acres, the groined arch roof will prove much more economical than the flat slab roof.

DISCUSSION

THOMAS H. WIGGIN,* M. AM. SOC. C. E. (by letter).—The groined arch of concrete in filter and reservoir roofs was fairly well standardized in the period from about 1898 to 1910, when the large slow sand filter plants at Albany, N. Y., Philadelphia, Pa., Washington, D. C., Pittsburgh, Pa., and Springfield, Mass., as well as a number of smaller plants, were being planned and built. Probably few engineers omitted the rather farcical process of drawing lines of resistance, but a crown thickness of 6 in. appears to have resulted in all cases except where unusual conditions existed, such as an additional structure to be supported by the groined roof. Spans, rises, and depth of depressions over piers were and are subject to considerable variation, the tendency being toward longer spans, and the author's tables and diagrams will be useful in showing the dimensions in many existing plants and the relative economy of various combinations of span, rise, and depth of depression over pier, as shown by data giving the average thickness of roof.

A table giving the dimensions of twenty-seven groined roofs, prepared by Leonard Metcalf, M. Am. Soc. C. E., was published in 1903.† This table was extended by Morris Knowles, M. Am. Soc. C. E., and, again, by the writer, and is included in the writer's paper entitled "The Concrete Groined Arch".‡ On page 181, the author refers to a condensed report of this paper. This table, as extended, gives data on forty-four groined arch roofs, about twelve of which are enumerated in the author's list of thirty in his Table 1. Data on about sixty-two roofs, therefore, are available in the two tables. The writer's table also gives data on floors for fifteen of the forty-four cases.

The author's Paragraph (*b*), on page 184, explains in general terms the effect of spacing and height of piers on the design of roof. It would be useful if he could include a table or diagram showing quantitatively the effect of span. The comparative tables and diagrams relate to a single span, namely, 17.96 ft., and the statement, on page 186, that "it will also be found that the average thickness decreases directly with the span", is not apparent from the examples from practice given in Table 1. Thus, compare Lines 21 and 30 of Table 1, as follows:

Line.	Clear span ($2a$), in feet.	Rise of intrados (b), in feet.	Crown thickness (t), in feet.	Rise of extrados (h), in feet.	Span, center to center of columns ($2c$), in feet.	Ratios.		Average thickness of roof, in feet.
						$\left(\frac{b}{2a}\right)$	$\left(\frac{h}{b+t}\right)$	
21	14.08	3.50	0.50	2.00	15.75	0.249	0.500	0.618
30	17.96	4.50	0.50	2.50	20.29	0.253	0.500	0.675

In Line 21, the pier capital is 1.67 by 1.67 ft., the area of which is one-eighty-eighth of the roof area; in Line 30, the pier capital is 2.33 by 2.33 ft.,

* Cons. Engr., New York City.

† *Journal*, New England Water Works Assoc., December, 1903, p. 400.

‡ *Proceedings*, National Cement Users Assoc., 1910, p. 246.

the area of which is one-seventy-sixth of the roof area. The relatively large pier capital in the case referred to in Line 30 evidently had more effect on the quantities per square foot than the span.

To make the comparison complete, quantities in piers should be included, as the author states; it is satisfactory to do so when the height of the roof at the crown is one of the fixed quantities, as is sometimes the case, as quantities in side-walls, in that case, are not affected by rise of the arch. In some cases, head-room near the piers or the assumption that the water level shall not be above the springing line, complicates the comparisons.

Quantities in floors should also be included in economic comparisons unless the reservoir is founded on rock so that the floor does not have to support the pier load. Analyses of stresses in groined floors are even less satisfactory than those in roofs.

In the interesting pursuit of theoretical economy, it is easy to forget some practical limitations. Thus, the depression over the pier, designated by h in the author's tables and diagrams, increasing the depth of which is so important in securing economy of material, is a trouble in construction. The deeper this depression relative to the span, the more difficult it is to keep the concrete from sloughing into the depression, also, the greater the chance of unintentionally robbing the arch of thickness at the haunch and the dryer the concrete must be. Furthermore, ramming on the slope makes the concrete slough into the depression. Forms for the depression have been tried, but only experimentally as far as the writer knows. Concrete workers on groined arches become surprisingly skillful in forming these depressions, yet it would doubtless be true economy to limit the steepness of the surface slope. It would be interesting to have a line of practical limitation (similar to the deflection limit in a table of floor-beams) added to the tables and diagrams on the basis of surface slope. A contractor at his own expense would generally prefer to fill exaggerated hollows somewhat. Probably, it would be better to assume a tangent of a certain slope at the lower end of the parabola of the extrados instead of extending the parabola to the bottom of the depression. The computation for quantity would be less easy. Allen Hazen, M. Am. Soc. C. E., described the depression in the roofs of the Albany filters as a cone and not a pyramid.*

The writer's paper, to which the author refers, contains this sentence: "The groined arch, although used centuries ago, remains a purely empirical type of structure, designed, or speaking more accurately, drawn, without satisfactory mathematical analysis and used without much knowledge of the factor of safety." The author takes a similar view and concerns himself with compiling aids for empirical design. The writer treated of certain other theoretical and practical problems in groined roof construction, that are related to design and, perhaps, may be briefly mentioned.

At the edges of a filter or reservoir, the thrust of a groined arch is often opposed by the thrust of a side-wall having a section about like one-half an ordinary horseshoe-shaped aqueduct. The "active" thrust from this side-wall is much less than that of the groined arch which is comparatively flat. Mani-

* *Journal*, New England Water Works Assoc., December, 1903, p. 408.

festly, the "passive" resistance of the earth against the side-wall is called into play, or the groined roof supports itself partly by cantilever action. Shrinkage and change of temperature of the roof are sufficient to cause the construction joints along the crown to open, sometimes as much as $\frac{1}{8}$ in. One who has designed arch bridges by rigid methods, in which the distortions due to stresses and temperature are consistently accounted for, knows that a short arch, even if it is without groins, could not take up such a shrinkage without cracking. When the ties afforded by the straight elements of the arches at right angles are considered, it is evident that, during the colder weather, it is only after the formation of vertical cracks along planes approximately joining piers that the groined roof can act as a rough voussoir arch. The writer has observed many such cracks. These cracks are not dangerous, unless they happen to occur on a slope such as to release a section of roof and allow it to fall. It is only this latter occurrence, which happens very infrequently, that would deter the writer from using arches 3 or 4 in. thick in some cases. The greater the rise, the less the danger that pieces may be released. Doubtless, the load carried by arch action through the earth covering, or, perhaps, by beam action in frozen earth, relieves the concrete of considerable stress.

It was formerly thought that groined arches could carry the load without arch action, that is, by cantilever action. The idea was suggested by the safety with which wagons and horse-drawn rollers could move near the free edges of partly constructed groined arch roofs. Computation of these loads, however, showed comparatively light concentrations. Furthermore, strips of roof were constructed monolithically and a traveling load would be supported by several groins aiding each other. Foremen's stories of long unsupported edges of groined arch roofs, covered nearly to the edge with full depth of earth fill, could never be substantiated by the writer. In order to obtain an idea of the cantilever strength of concrete groined roofs where they are not supported by arch action, several isolated groin units, symmetrical about a pier, were built and tested in 1905 by the Bureau of Filtration of Pittsburgh, Pa., of which Morris Knowles, M. Am. Soc. C. E., was then Chief Engineer. The work was done under the direction of the writer, then Division Engineer. W. A. Bassett, M. Am. Soc. C. E., was Assistant Engineer in immediate charge of the construction and testing. These tests have never been published and by arrangement with Mr. Knowles are presented subsequently as a part of this discussion. Most of the data are from Mr. Bassett's final report dated February 13th, 1906.

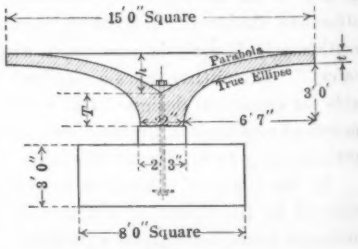


FIG. 5.

Nine groin units, each 15 ft. square, were tested. Fig. 5 and Table 5 give the dimensions, breaking loads, and deflections of these groin units. Fig. 6 gives sketches of the lines of fracture for Groins Nos. 1, 2, 4, and 6. Other data are as follows.

TABLE 5.—DIMENSIONS AND TESTS OF 15 BY 15-FT. GROIN UNITS.

Groin No.	Thickness at crown, in inches.	Thickness over pier, *t*, in inches. ‡	Age, in days.	Deflection at edge, in inches. §	Breaking Load Exclusive of Dead Weight and of Sand Used in Leveling.	
					Total, in pounds.	Pounds per square foot.
1	6	21	41	0.002	19 585	88
2	6	21	39	0.002	53 985	240
3	4½	19½	Broke on removal of form. Concrete 6 days old.			....
4	3	18	Broke on removal of form. Concrete 18 days old.			....
5	6	21	39	0.000	13 000	58
*6	6	21	35	0.005	50 700	225
7	6	21	40	0.004	30 000	133
*8	6	21	40	0.005	39 900	178
†9	6	21	40	0.003	33 300	148

* Reinforced with Thacher rods.
† Aggregate composed of sand and blast-furnace slag, proportions, 1 : 2¼ : 5¾.
‡ Rise of extrados, *h*, is 21 in., or 1.75 ft. for all nine groins.
§ Deflections are not regarded as very reliable. They do indicate that deflection was small.

Foundations.—Blocks of 1:3:7 concrete, 8 ft. square and 3 ft. thick, were built in excavations made in the fill of blast-furnace slag at the "Isabella" furnaces near Pittsburgh. The fill was firm, but subject to slight jar from trains passing on adjacent freight tracks.

Dimensions of Groin Units.—The bearing on piers was 22 in. square, leaving a clear semi-span of 6 ft. 7 in., so that the full span would be 13 ft. 2 in. The rise was 3 ft. in all cases. These dimensions were the same as those used in constructing the filter plant and are given in Line 14 of the author's Table 1. The crown thickness was in general 6 in., but Groins Nos. 3 and 4 had thicknesses of 4½ in. and 3 in., respectively. The depth of depression, or rise of extrados as the author names it, was 1.75 ft., but the bottom of the depression was filled in slightly to a flat surface to make a bearing for the anchor-bolt.

Groins Nos. 6 and 8 were reinforced with a single ¾-in., Thacher reinforcing bar, which was placed near the outer edge at about mid-height in the 6-in. thickness. It was, therefore, in the form of a square, but the corners were rounded. (See plan of fracture of Groin No. 6 on Fig. 6.)

Cement Used in Groin Units.—Lehigh cement was used and the standard tests of briquettes gave the results shown in Table 6.

Aggregates Used in Concrete for Groin Units.—In general, sand and gravel dredged from the Allegheny River were used. In one case, namely, Groin No. 9, the gravel was replaced with blast-furnace slag, a greenish, vitreous, somewhat porous material, much lighter than gravel. Allegheny River sand is well graded and not too fine. The writer, unfortunately, has no records of the mechanical analyses available. The gravel is sound and hard, but rather flat, and is well graded from fine to coarse, the larger stones being about 1½ in. across.

Quality of Concrete in Groin Units.—The concrete was mixed by hand, but thoroughly. Its strength is indicated by the tests given in Table 7, of beams, 6 in. square in section, made with concrete mixed and ready to place in the groins.

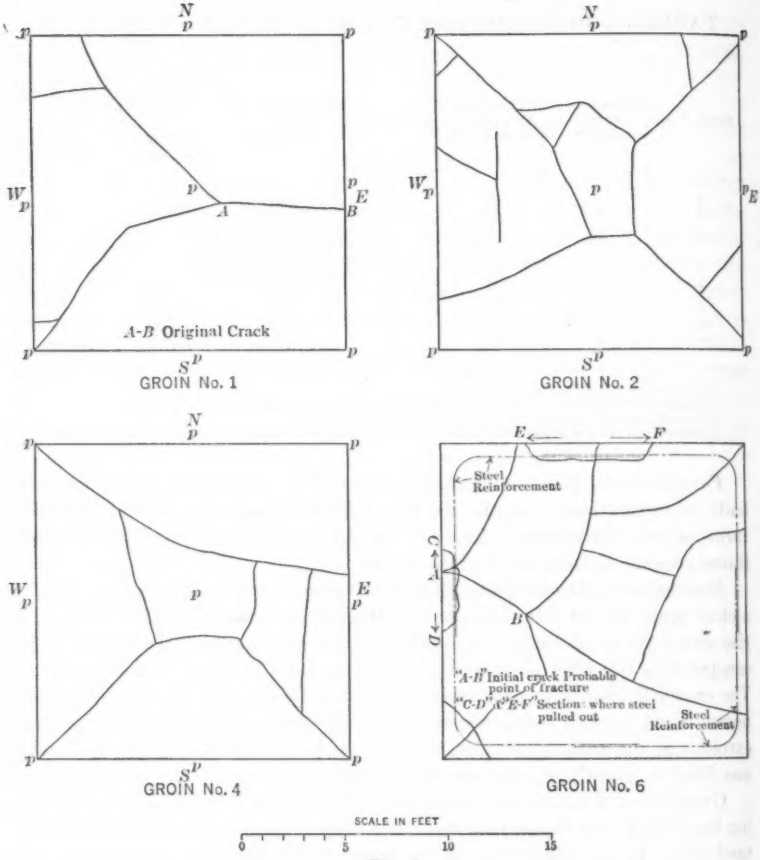


FIG. 6.

TABLE 6.—TESTS OF CEMENT.

	NEAT.		1 : 3	
	7 days.	28 days.	7 days.	28 days.
SAMPLES REPRESENTING WHOLE SHIPMENT OF 145 BBL.:				
Average	735	869	260	350
Maximum	1 025	1 065	305	395
Minimum	365	675	215	290
SAMPLES FROM CEMENT USED IN PARTICULAR GROINS:				
Groin No. 1	.....	.....	200	280
" " 3	750	900	175	300
" " 5	700	700	225	325
" " 8	620	563	223	305
" " 8	706	625	233	281

Groins Nos. 1 to 8 were made of 1:3:5 sand and gravel concrete, and Groin No. 9 was made of 1 part cement to 2½ parts sand to 5¾ parts blast-furnace slag. Much water was required for this latter mix, possibly on account of the porosity of the slag.

TABLE 7.—TESTS OF 6 BY 6-IN. BEAMS MADE FROM CONCRETE MIXED FOR GROIN UNITS.

Concrete from Groin No.	Age, in days.	Span, in inches.	Center load, in pounds.	Modulus of rupture, in pounds per square inch.
1	287	30	1 260	264
2	294	30	2 598	557
2	297	60	1 098	502
3	299	30	2 653	581
3	309	30	2 506	502
3	312	60	903	401
5	220	30	1 101	253
5	220	60	382	194
6	196	30	1 247	258
7	195	30	1 376	294
8	163	30	1 474	314
9	180	30	1 003	213

The groin units probably did not set as rapidly as they would have in general in the filters, because there was no drain from the depression and the early summer rains kept them filled with water much of the time. The concrete, however, appeared to be fairly well set and strong.

Tests of Groin Units.—Slag was piled under the groins leaving a space of only about 3 in., in order that the impact of the fall might not break them into smaller pieces and disguise the lines of fracture produced by the loading. In spite of this precaution, it is known that many of the cracks shown on the diagrams of fracture, in Fig. 6, resulted from the fall. The fall was difficult to watch, but it is known that the initial break in nearly all cases took place near the center of a side. However, Groin No. 2 which was the strongest, apparently broke first nearer a groin line than the center of a side. Groins Nos. 1 and 5 cracked before any load was applied, owing perhaps to shrinkage stresses or jarring, or to a combination of these causes. Final failure took place at the same cracks under a light loading. Groins Nos. 3 and 4, respectively 4½ and 3 in. thick at the crown, fell down when the forms were removed, which was done at 6 days and 18 days, respectively.

Iron, in pigs weighing about 100 lb. each, was used in the loading. These pigs were piled corncob fashion on isolated wooden bases, 22 by 24 in., symmetrically placed on the level bed of sand with which the depression in the groin had been previously filled. Therefore, no tying strength was afforded by the load itself.

It appears to be a safe conclusion that groins made of good concrete and supported on piers about 15 ft. apart, will about sustain a fill of 2 ft. of earth without developing arch action. As the groins are generally concreted in strips, the strength of part of the roof would be greater, owing to the effect of continuity.

The effect of steel reinforcement was not fairly measured by tests of Groins Nos. 6 and 8, because the failure took place in the region of the lap, and the steel pulled out. In Groin No. 8, the steel was flattened, drilled, and fastened together at the splice with a number of ⅞-in. bolts. The steel broke at a bolt hole. It would probably have been better to put the laps at the corners, although the best groin—No. 2—broke at and near the corners. Computations indicate that such a small quantity of steel would not be very effective in any case. Cantilever action of groined roofs is unnatural, being inconsistent with arch action. Steel placed to aid cantilever action would be of use only to take construction stresses occurring near unsupported edges. There are better ways to provide for these stresses, as explained at length in the writer's paper referred to previously.

The author's statement on page 189 recommending concrete mixes not less rich than 1:2.5:5 is a little over-conservative, the writer believes, as most groined roofs have been built of 1:3:5 concrete, or of concrete of equivalent richness.

The author's statements as to the relative economy of the groined arch and reinforced concrete slab roofs are doubtless correct. The writer's estimates of relative cost, also contained in the paper referred to, showed that the reinforced concrete beam and slab roof costs about double that of the groined arch roof on works sufficiently large to permit repeated use of the forms.

PHILIP O. MACQUEEN,* ASSOC. M. AM. SOC. C. E. (by letter).—The results obtained by the tests, made in 1905 by the Bureau of Filtration of Pittsburgh, Pa., on isolated groin units form a valuable contribution to the subject. Tests on full-sized groins, or even models of groins, are scarce, so that the data now published for the first time in the discussion by Mr. Wiggin are particularly welcome. The results given by him in Table 5 indicate with a fair degree of clearness the general cantilever strength of a typical groined arch, and it is observed that this strength is sufficient to sustain approximately the usual full dead load of 2 ft. of earth, which the average arch is called on to bear. The failure of the arches having relatively thin sections at the crown is deserving of especial attention, as well as the general occurrence of the initial break near the center of the sides. Cantilever action, as noted by Mr. Wiggin, is unnatural in groined arch roofs, but, nevertheless, it forms an important factor of safety in many cases where joints in roofs have opened slightly due to contraction.

Mr. Wiggin has called attention to the statement made by the writer that "it will also be found that the average thickness decreases directly with the span," and has shown that this statement does not necessarily follow from the results given in Table 1. It is true that the change in average thickness of roof shown in Table 1 does not directly follow the change in span, but this is due to the personal element which has entered into the various designs. For example, it will be noted that the average roof thickness for Arches Nos. 8 and 9 are high, principally on account of the very small depressions over the columns which the designers have used in these special cases. Other sudden

* Cons. Municipal and San. Engr., Washington, D. C.

changes in average roof thickness due to somewhat similar reasons will also be observed. However, there is a general tendency for the average roof thickness to decrease with the span.

The effect of change of span on the total quantity of concrete is difficult to determine and can only be shown by making certain assumptions. Table 8, therefore, furnishes data on a set of imaginary arches in which the dimensions are made to conform in general with those recommended by the writer on page 189. It has been assumed that all the columns are square with special bases omitted and that the height of the reservoir from the floor to the under side of the roof at the arch crown is 18 ft. It has also been assumed that the floor of the reservoir is level and of the same thickness throughout.

It will be observed from Table 8, as stated by the writer on page 186, that the average roof thickness (Column 12) decreases almost directly with the span and that the total average thickness of roof plus column (Column 14) also decreases slightly with the span.

On account of its important bearing on the calculation of total quantities of concrete, the writer is glad that Mr. Wiggin has raised the question of the effect of span. Table 8 must be considered as lacking more or less in definite indications, as the arches are assumptions on the part of the writer, but it seems fairly safe to conclude that the change of span has relatively small effect on total quantities. As the span decreases, the volume of concrete in the roof decreases, but as the volume in the columns increases, the total volume remains practically the same. The principal saving, therefore, in using longer spans would be in form work, with the additional advantage of increased capacity in the reservoir.

Comparing Arches Nos. 21 and 30, Mr. Wiggin has called attention to the effect of the size of the pier capital on the average roof thickness. Making the pier capital for Arch No. 30, one-eighty-eighth instead of one-seventy-sixth of the roof area would decrease the average roof thickness from 0.675 to 0.663 ft. This is a decrease of only 2%, whereas the change in average roof thickness due to decrease of span from Arch No. 30 to Arch No. 21 is from 0.675 to 0.618 ft., or about 8 per cent. It appears to the writer, therefore, that in this case the change of span has greater effect on the quantities than the change of size of the pier capital.

As stated by Mr. Wiggin, it is absurd to carry the subject of theoretical economy to the extreme, and there are practical limitations in groined arches as well as in all other construction work. Considering the structure outlined in his paper, the writer would recommend the average economy which ordinarily is used in designing concrete work, for example, such as beams, columns, and retaining walls.

The steel reinforcement for groined arch roofs is a question that is probably discussed each time a new roof is designed. In practice, reinforcement is seldom used, although in certain instances the writer believes it is advisable, such as in the new Baldwin Reservoir for the City of Cleveland, Ohio,* in

* *Engineering News-Record*, November 30, 1922, p. 916.

TABLE 8.—DATA SHOWING EFFECT OF SPAN ON QUANTITIES OF CONCRETE IN GROINED ARCH RESERVOIR CONSTRUCTION.

No.	Clear span ($2a$), in feet.	Rise of intrados (b), in feet.	Crown thickness (t), in feet.	Rise of extrados (h), in feet.	Span center to center of columns ($2c$), in feet.	Thickness of square column ($2c$-$2a$), in feet.	Assumed height of reservoir roof above floor, in feet.	Clear length of column, in feet.	Volume in roof groin, in cubic feet.	Volume in column, in cubic feet.	Average thickness of roof groin, in feet.	Equivalent thickness of roof groin for volume in column, in feet.	Total average thickness (roof plus column), in feet.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	10.0	1.667	0.5	1.083	11.250	1.250	18.0	16.333	67.969	25.528	0.537	0.202	0.739
2	11.0	2.000	0.5	1.250	12.333	1.333	18.0	16.000	83.701	28.432	0.553	0.187	0.740
3	12.0	2.333	0.5	1.416	13.417	1.417	18.0	15.667	101.169	31.459	0.562	0.175	0.737
4	13.0	2.667	0.5	1.583	14.500	1.500	18.0	15.333	121.186	34.499	0.576	0.164	0.740
5	14.0	3.000	0.5	1.750	15.583	1.583	18.0	15.000	142.972	37.590	0.588	0.155	0.743
6	15.0	3.333	0.5	1.916	16.667	1.667	18.0	14.667	167.074	41.860	0.603	0.147	0.750
7	16.0	3.667	0.5	2.083	17.750	1.750	18.0	14.333	193.425	43.782	0.614	0.139	0.753
8	17.0	4.000	0.5	2.250	18.833	1.833	18.0	14.000	222.043	47.040	0.625	0.132	0.757
9	18.0	4.333	0.5	2.416	19.917	1.917	18.0	13.667	249.796	50.226	0.628	0.126	0.754
10	19.0	4.667	0.5	2.583	21.000	2.000	18.0	13.333	286.943	53.332	0.650	0.121	0.771

which the columns and spans are of unusual length. In the Baldwin Reservoir, single, straight, ¾-in. steel rods were placed along the perimeter of each groin adjacent to the crown, and the rods used were of such length that four groins in a group were joined together. Adjacent groups of four groins were not connected. The reinforcement was intended more for construction stresses and construction safety than for working stresses.

In the discussion, attention is called to the use of groined arches in the floors of reservoirs. The writer is not in favor of using groined arches in this manner. Plain floors will be easier to place if the foundations are solid, and they can be designed so as not to involve extra concrete. If the foundations are not solid, the groined arches would be likely to open due to slight settlement and cause very heavy leakage.

The proper concrete mixture for groined arches is a subject for the most careful consideration, and the writer must admit his conservativeness on this question. The decision to use certain proportions does not guarantee a predetermined strength, as the quality and size of the aggregate and other factors are of equal or, perhaps, more importance. If possible, the proportions should be determined from actual tests. In the writer's opinion, the average ultimate compressive strength of the concrete developed in 28 days should not be less than 1 800 lb. per sq. in.

In conclusion, the writer wishes to state again that the various comparisons and approximate formulas discussed in his paper are intended principally as an aid in the design of concrete groined arches, and it is believed that the points noted in the paper and in the thorough discussion by Mr. Wiggin will be of assistance in this special field.

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Paper No. 1512

TENTATIVE PLAN FOR THE CONSTRUCTION OF A 780-FOOT ROCK-FILL DAM, ON THE COLORADO RIVER, AT LEES FERRY, ARIZONA

By E. C. La Rue,* M. Am. Soc. C. E.

With Discussion by Messrs. C. R. F. Coutlee, H. B. Muckleston, Edwin H. Warner, F. A. Noetzli, Kirk Bryan, Arthur P. Davis, J. C. Stevens, J. H. Quinton, Ernest H. Baldwin, C. S. Jarvis, Samuel Fortier, L. F. Harza, I. Gutmann, and E. C. La Rue.

Synopsis

It is proposed to build a rock-fill dam in a narrow canyon of the Colorado River by blasting in the canyon walls. The dam described in this paper was designed to raise the water 700 ft. above the bed of the river. Both the height of the dam and the plan suggested for its construction are unique features. The walls of the canyon rise 1 300 ft. above the river, and the width of the canyon at the water surface is 450 ft.

During the past six years, the writer has had in mind the plan herein outlined for the construction of such a dam. There may be a difference of opinion as to the feasibility of the plan as presented. Some engineers may have studied this problem carefully, but to the writer's knowledge no one has ever made the results of his studies available to the Engineering Profession. It is hoped that engineers will present their views in the form of constructive criticism of this paper.

Introduction

In the southwestern part of the United States, the demand for power is increasing rapidly. In California alone, construction plans which call for

* Hydr. Engr., U. S. Geological Survey, Pasadena, Calif.

the expenditure of several hundred million dollars, have been adopted for the development of hydro-electric power. As a result of these plans, the ultimate development of the power resources of the Sierra Nevada and Coast Range Mountains, in the southern part of California, is in sight.

The construction of high rock-fill dams by blasting down the towering canyon walls has been suggested in connection with studies for both power and irrigation development. This paper deals primarily with the practicability of constructing such a dam in the Colorado River. A particular site (Lees Ferry, Ariz.) has been chosen, in order that the discussion might be based on definite dimensions. The paper should not be construed, however, as advocating this or any other specific project.

Colorado River Basin.

General Features.—The Colorado River is formed by the junction of the Green and Grand Rivers in Southeastern Utah. The combined length of the Green and the Colorado is 1 700 miles and the total fall is about 14 000 ft.

The Colorado River and its tributaries drain an area of 244 000 sq. miles. This area comprises the southwestern part of Wyoming, the eastern half of Utah, the western part of Colorado, practically all of Arizona, and small parts of California, Nevada, New Mexico, and Mexico. (Fig. 1.)

The water resources of the Colorado River Basin are enormous. About 14 000 000 acre-ft. of water is wasted annually into the Gulf of California. Practically all this water could be used for irrigation and, in addition, a large amount of hydro-electric power could be developed. In order to utilize fully the water resources of the Colorado River Basin, the flow must be regulated to meet the demands for both irrigation and power. The storage of the flood waters would serve a threefold purpose: Water would be available for irrigation when needed; the possibilities for the development of hydro-electric power would be greatly increased; and with the ravaging floods under control, property on the lower river, exceeding $100 000 000 in value, would be safe.

Storage Reservoir Sites.—A comprehensive plan for the development of the water resources of the Colorado River Basin will undoubtedly call for the utilization of most of the reservoir sites mentioned in Table 1. The locations of these sites are shown on Fig. 1.

Table 1.—Storage Reservoir Sites in the Colorado River Basin.

Name.	Stream.	Height of dam, in feet.*	Capacity, in acre-feet.
Flaming Gorge	Green River, Utah-Wyoming	255	4 720 000
Juniper Canyon	Yampa River, Colorado	200	1 400 000
Ouray	Green River, Utah	300	28 000 000
Kremmling	Grand River, Colorado	230	2 200 000
Dewey	Grand River, Utah	213	2 500 000
Green-Grand	Junction of Green and Grand Rivers, Utah	270	8 600 000
Bluff	San Juan River, Utah	264	2 600 000
Lees Ferry	Colorado and San Juan Rivers, Utah-Arizona	700	50 000 000†
Boulder Canyon	Colorado River, Nevada-Arizona	600	31 600 000

* The figures in this column indicate the elevation of spillway above river bed.
† Estimated.

These storage sites and their use were discussed in the writer's report on the "Colorado River and Its Utilization".* An analysis was made of the records of stream flow, and mass curves indicated that the flow of the Lower Colorado could be equalized if properly located reservoir sites having an aggregate storage capacity of 25 000 000 acre-ft. were utilized. This study also indicated that, if no allowance is made for evaporation from the surface of these reservoirs, a uniform flow of 21 800 sec-ft. could be maintained below the San Juan River. The irrigation of additional lands in the Upper Basin would further reduce the quantity for the uniform flow that might be maintained by storage.

Undeveloped Water Power Between Green River and Boulder Canyon.—In the 670 miles between the Town of Green River, Utah, and Boulder Canyon, Nevada-Arizona, the fall is about 3 350 ft. As this part of the Colorado and that part of Grand River below the elevation of the Town of Green River are in practically a continuous canyon, the use of the water for irrigation within this stretch is impossible; but conditions are favorable for the regulation of flow by storage and for the development of power. The entire regulated flow might profitably be used for the irrigation of extensive areas of arable land below Boulder Canyon.

The data now available indicate that the water power resources of the 670 miles of the river between Green River and Boulder Canyon, may be utilized by developing five projects.

Green-Grand Project.—A 170-ft. dam built on the Colorado River immediately below the mouth of Green River would make possible the development of 110 000 h. p. The capacity of this project for power may be greatly increased when the flow of the Green and Grand Rivers is regulated by additional storage in the upper basin of these streams.

Lees Ferry Project.—By building a dam at Lees Ferry to raise the water 700 ft., 1 000 000 h. p. may be developed.

Marble Canyon Project.—Assuming that the flow of the river is regulated by storage above, it may prove feasible to develop 1 380 000 h. p. between Lees Ferry and the western boundary of the Grand Canyon National Park.

Diamond Creek Project.—Between the western boundary of the Grand Canyon National Park and Diamond Creek, the Colorado River falls 500 ft. This part of the river may be developed by building two dams, each dam to raise the water 250 ft. The capacity of this project would be 830 000 h. p., if the flow of the river is regulated by storage above.

Boulder Canyon Project.—It is assumed that at Boulder Canyon the flow will be re-regulated to meet the demand for water for irrigation. Under this condition, a dam 550 ft. high will make possible the development of 700 000 h. p.

More than 4 000 000 continuous horse-power may be developed between Green River and Boulder Canyon. The aggregate installed capacity of the power plants would probably be more than 5 000 000 h. p.

The Lees Ferry Project

Eight miles above Lees Ferry, Ariz., is a site which has been selected as particularly favorable for both the regulation of flow and the development of

* *Water Supply Paper No. 595*, U. S. Geological Survey, 1916.

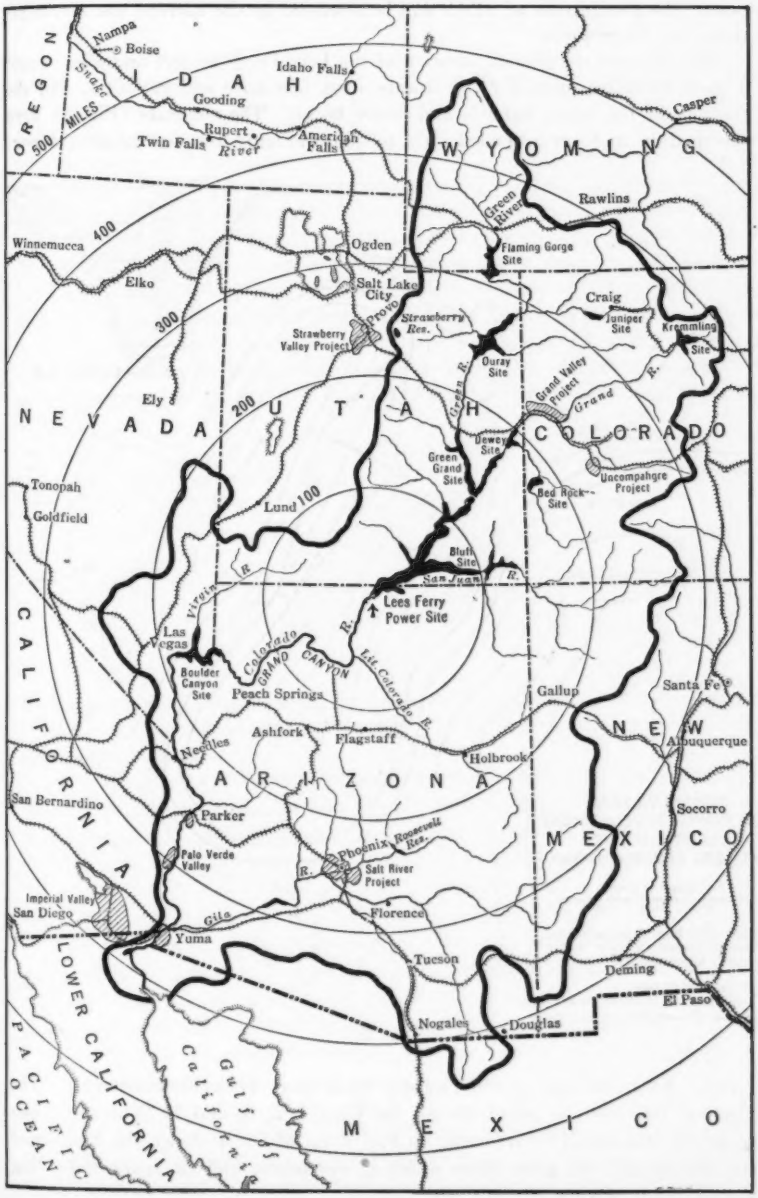

OREGON
Nampa
Boise
Snake
MILES
500
IDAHO
Idaho Falls
Gooding
Rupert
Twin Falls
River
American Falls
WYOMING
Casper
Rawlins
Green River
Flaming Gorge Site
Winnemucca
400
Ogden
Elko
Salt Lake City
Provo
Strawberry Res.
Strawberry Valley Project
300
Craig
Juniper Site
Kremmling Site
Ouray Site
Uinta R.
Grand Valley Project
Grand R.
Ely
200
NEVADA
UTAH
COLORADO
Dewey Site
Green Grand Site
Uncompahgre Project
Bed Rock Site
Tonopah
Goldfield
Lund
100
Bluff Site
San Juan R.
Virgin R.
Lees Ferry Power Site
Las Vegas
Colorado R.
GRAND CANYON
Lit. Colorado R.
CALIFORNIA
Boulder Canyon Site
Peach Springs
Gallup
Santa Fe
Needles
Ashfork
Flagstaff
Holbrook
NEW MEXICO
Albuquerque
ARIZONA
San Bernardino
Parker
Socorro
Palo Verde Valley
Phoenix
Roosevelt Res.
Salt River Project
Imperial Valley
San Diego
Gila R.
Florence
Yuma
Tucson
Deming
El Paso
Nogales
Douglas
LOWER CALIFORNIA
PACIFIC OCEAN
Gulf of California
MEXICO

FIG. 1.

power, the possibilities of which will be disclosed by the surveys and investigations now in progress.

Fig. 1 shows the general location of the Lees Ferry project and Figs. 2 and 3 show tentative plans for the storage dam, alternate spillway sites, and the position of the power tunnels and power house. The contours (Fig. 2) were sketched in and are intended only to show roughly the surrounding topog-

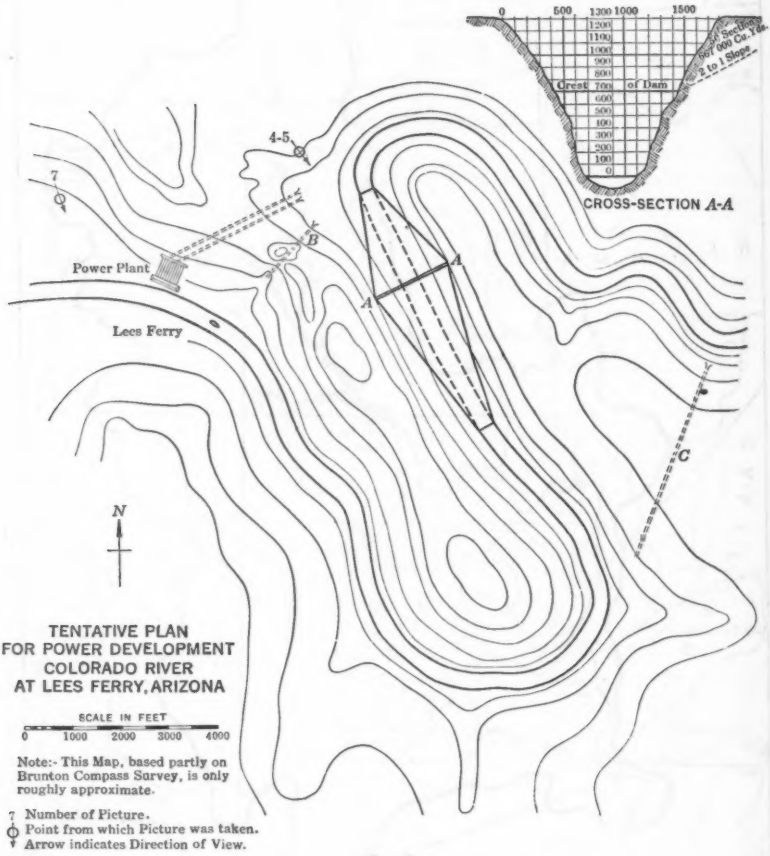


FIG. 2.

raphy. A careful survey will probably show more favorable conditions. The physical features are shown clearly in Figs. 4, 5, 6, and 7. The views can be better understood by referring to Fig. 2, on which is shown the number of the photograph, the point from which it was taken, and an arrow indicating the direction of view.

Present information indicates that to construct a 780-ft. dam at the Lees Ferry site may prove feasible. A dam of this height would form a reservoir with a capacity of about 50 000 000 acre-ft. The maximum back-water would extend up stream nearly to the junction of the Green and Grand Rivers and to within 230 ft. of the elevation of the piers of the railroad bridge near Green River, Utah.

The capacity of the upper 100 ft. of the Lees Ferry Reservoir would be about 20 000 000 acre-ft. Allowing for evaporation from the reservoirs and for the use of water in the upper basin for the irrigation of additional lands, the 20 000 000 acre-ft. of storage capacity may be sufficient to equalize the flow through the canyons of the lower river. With a 100-ft. drawdown, the average head available at Lees Ferry for power would be about 650 ft., and with the flow of the river equalized at this point, power plants in the canyon below could be operated under heads almost equal to the full height of the respective dams. The dam farthest down stream, however, should provide sufficient storage capacity to regulate the flow to conform to the demand for irrigation.

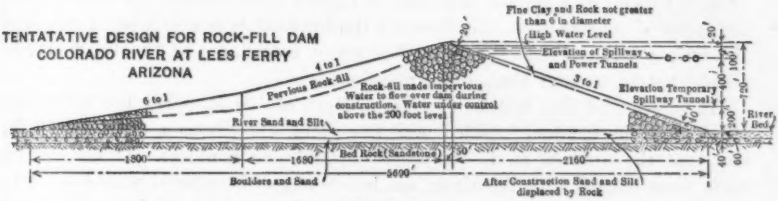


FIG. 3.

Under these conditions, it seems probable that an aggregate effective head of 2 800 ft. may be utilized out of the 3 350 ft. of fall between Green River, Utah, and Boulder Canyon.

The mean annual flow at Lees Ferry has been about 15 000 000 acre-ft., but allowing for the reduction in flow for ultimate irrigation development in the upper basin, this quantity may be reduced to 10 000 000 acre-ft. It may be fifty years before ultimate irrigation development in the upper basin is reached. To assume that a uniform flow of about 19 300 sec-ft. can be maintained at the Lees Ferry site and in the canyons below would seem reasonable.

The depth to bed-rock at the dam site has not been determined, but the writer believes that rock is near the surface in the river channel. When he examined this part of the river in August, 1915, the river was at a very low stage, and down stream about 8 miles from the Lees Ferry dam site, the bed-rock appeared to extend across the channel of the Colorado River at the surface.

At the dam site, the red sandstone walls of the canyon rise 1 300 ft. above the river, and the width of the canyon at the water surface is 450 ft. The course of the river at this point is in the form of the letter "S", thus making two spillway sites available. By constructing tunnels through the bluff, the water may be carried about 3 000 ft. to a forebay and power house which is 6 miles by river from the dam site. The site is easily accessible by automobile

and a railroad could be built at a comparatively low cost, connecting with the nearest railroad point, Flagstaff, Ariz., 133 miles to the south. After considering the conditions at the dam site, the facilities for spillways, site for power house and auxiliary structures, storage capacity above the dam, and the accessibility of the project, the writer believes the Lees Ferry site to be one of the most promising on the Colorado River below the junction of the Green and Grand Rivers.

Should this project be developed, water would be available for irrigation on the lower river for at least a generation to come, and the menace from floods would be removed (except floods from the Gila River).

The conditions favor the construction of a 780-ft. dam of the rock-fill type. (Fig. 3.) Although such a high rock-fill dam has never been constructed, the writer believes that one can be built at this site and made practically watertight at a much less cost than any other type of dam.

Tentative Design for Rock-Fill Dam.—In designing a rock-fill dam for the Lees Ferry site, a slope of 3:1 was used for the up-stream face; for the down-stream face, 6:1 was used to the 300-ft. level, and 4:1 for the remainder. (Fig. 3.) The rock for the dam can be blasted from the canyon walls. With this plan of construction, the slopes of the finished dam would be uneven, but the writer believes that the general slope of both faces of the finished dam should not be greater than those mentioned.

It is assumed that bed-rock lies at a depth of 100 ft. below the bed of the river, that the channel is filled with fine sand and silt to a depth of 60 ft., and that the next 40 ft. is composed of sand and boulders. If the canyon walls were blasted into the river, the heavy mass of material would settle to the level of the boulders assumed to be 60 ft. below the bed of the river, and some of the fine sand and silt would be forced up. Under these conditions, the finished dam would contain 50 000 000 cu. yd. of material. The center 50-ft. section of the dam would contain about 1 000 000 cu. yd.

Prior to construction, the outline for the dam could be plainly marked on the canyon walls. (Fig. 4.) A plan for mining both walls of the canyon could be worked out from detailed topography of the site of the dam. To obtain 500 000 cu. yd. of rock from each wall for the center 50-ft. section of the dam, it would be necessary to blast the walls back on a slope slightly steeper than 2:1 (Fig. 2), and, here, it may be necessary to rehandle some of the material. Up and down stream from the center 50-ft. section of the dam, the walls of the canyon may be blasted from a lower level, and the required quantity of material obtained by blasting back on a much steeper slope. The space occupied by the loose rock in the dam will be about 15% greater than the space occupied by the same material in its natural compact state. If the mines are properly placed in both walls of the canyon, the first shots will blow enough rock into the river to form a dam 250 ft. high, having a gradual slope to the lower toe, more than a mile down stream.

The discharge of the river may be expected to range from 4 000 to 10 000 sec-ft. during the eight months following the flood period. Perhaps 50% of the material in the dam would consist of rock weighing from 1 ton to 1 000 tons. If the unexpected should happen, and a flood of 100 000 sec-ft. should



FIG. 4.—DOWN-STREAM VIEW OF DAM SITE AT LEES FERRY, ARIZ.

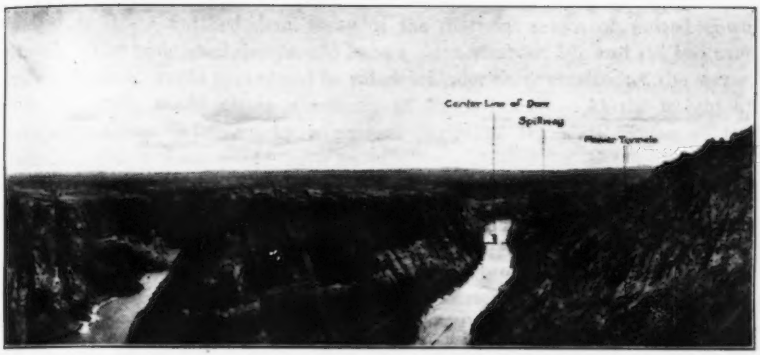


FIG. 5.—PANORAMIC VIEW, LEES FERRY, ARIZ.



FIG. 6.—UP-STREAM VIEW, 1 MILE BELOW DAM, LEES FERRY, ARIZ.



FIG. 7.—VIEW SHOWING SITE OF POWER HOUSE, ETC., LEES FERRY, ARIZ.

pass over the unfinished dam, some of the material might be moved down stream. The unfinished dam would have a slope of about 5% and the bed over which the flood would pass would be rough. Under these conditions, the water, 8 ft. in depth, might attain a velocity of 30 ft. per sec. As the weight of bodies that can be moved by a current varies as the sixth power of the velocity, such a flood would have an enormous ability for destruction. The lighter materials would be carried down stream, and rock that could withstand the force of the water would settle. However, floods which occur outside the normal flood period, last only a few days. To believe that after the flood, the unfinished dam would retain a height of 100 ft. or more above the average low-water level seems more reasonable than that all of it would be carried away. Such a flood might prove to be a blessing in disguise, as that part of the dam remaining would form a solid foundation on which to build the superstructure.

However, flood water should not be permitted to flow over the dam, and it is suggested that no material be blasted into the river until a temporary spillway has been provided 200 ft. above the bed of the river. A spillway tunnel located at *B* (Fig. 2), would be about 3 000 ft. long and could be given a fall of 200 ft. A spillway tunnel, 30 ft. in diameter, with a properly designed entrance, would have approximately the following capacity:

Elevation of water above gate-sill, in feet.	Capacity, in second-feet.
0	0
10	5 000
20	13 500
30	22 500
40	28 500
50	33 700
60	38 200
70	42 000
80	45 000
90	47 400
100	50 000

With the spillway completed and the flood stage passed, the mines may be discharged and about 20 000 000 cu. yd. of rock would be blown into the river. This mass of loose rock would not be water-tight which feature is the most uncertain in the plan here presented.

This mass of rock, weighing about 40 000 000 tons, would displace the fine sand and silt and would tend to form a mud bank immediately up stream from the dam. Such a mud bank would be practically water-tight and, until overtopped, it would cut off the flow of the river. This material might make the rock-fill water-tight to an elevation of 50 to 100 ft. above the bed of the river, and thus form a reservoir having a storage capacity of from 100 000 to 700 000 acre-ft. If the flow of the river was 5 000 sec-ft., the supply of water for irrigation might be cut off for a period of from 10 to 70 days.

Water for the irrigation of more than 400 000 acres of land on the Lower Colorado must be available each month of the year. To cut off the supply of water for the irrigation of these lands, even for a 10-day period, might result in serious damage to crops, and an understanding should be had with the irrigation interests, in order that they may take proper steps to protect the crops should the supply be cut off.

Therefore, unless a temporary spillway is provided at an elevation of 50 ft. above low water, it will be necessary to discharge the mines when the river is in flood so that the storage reservoir above the dam may be quickly filled, and permit the water to flow either through the upper part of the unfinished dam or through a temporary spillway located at the 200-ft. level. The writer believes that the temporary spillway should be located at the 200-ft. level and that the mines should be discharged after the peak of the summer flood, when the discharge has fallen to about 50 000 sec-ft. This feature of the project deserves consideration, and a thorough study of the stream-flow records should be made to determine the proper time for the first discharge of the mines.

Practically all the rock-fill can be blasted into place, thus making it unnecessary to rehandle the material except for finishing a small section at the top of the dam. The 50 000 000 cu. yd. of rock could probably be blasted into place at a cost of 10 cents or less per cubic yard. To determine the quantity of fine material necessary to make the rock-fill water-tight would be difficult, but the writer believes that about 10 000 000 cu. yd. of fine rock, sand, and silt would be required.

The upper face of the dam would be permanently submerged to the 600-ft. level, and, therefore, below this level no protection against wave action is necessary. The water in the reservoir would fluctuate between the 600 and 700-ft. levels. The reservoir would be located in a deep box canyon the circuitous course of which would prevent excessive wave action. (Fig. 3.) On the upper face of the dam, above the 600-ft. level, rock 2 ft. in diameter should probably be placed. This section will be wet and dry periodically and will require care in its construction, if the back-fill is to be kept water-tight.

Making the Rock-Fill Water-Tight.—Several methods have been used in the past to make rock-fill dams water-tight, but most of these methods appear to be unsatisfactory when applied to such a high dam.

A concrete core-wall extending to bed-rock is often used. If this plan were adopted at the Lees Ferry dam site, it would be necessary to excavate the river channel to bed-rock. To take care of the flood water during the period when the core-wall is being built from bed-rock to the former water surface, would be both hazardous and expensive. For such a high dam there would be danger of the core-wall becoming fractured on account of uneven forces being exerted by the rock-fill.

A steel core, having a concrete base, would be even more objectionable than a concrete core, for besides requiring a concrete facing to prolong its life, it would have all the objectionable features of the concrete core.

The rock-fill might be made water-tight by lining the up-stream face of the dam with concrete, but it would be necessary either to carry this concrete facing to bed-rock or to provide an impervious base for the dam with a series

of cut-off walls. Otherwise, the water would pass under the concrete facing and undermine the dam. In addition to these objectionable features, the settlement of the rock-fill would make it almost impossible to prevent the concrete facing from becoming fractured.

The writer believes that, with the river under control, the rock-fill can be made water-tight by sluicing fine material into the dam.

The discharge of the river ranges from 4 000 to 10 000 sec-ft. after the flood period. This quantity of water might find its way through the rock-fill. At this stage of construction, every attention should be given to the work of making the rock-fill water-tight. It would be necessary to place fine material on the upper face and perhaps as far down stream as the center of the finished dam. This material should be graded in size from that which would pass through a 1-in. to a 6-in. mesh. On the right bank of the river, a few hundred feet up stream from the upper toe of the dam, there is a talus which could be moved to the dam at a low cost. The back-water above the dam might have a depth of from 75 to 100 ft. and would extend up the river a distance of at least 25 miles.

Assuming that the rock-fill rises 250 ft. above the bed of the river, and that there is 100 ft. of water above the dam, the flow of water through the dam would be as follows: The water would rush through the open spaces between the large rocks, flow through the spaces between the smaller rocks, percolate through the sand, and seep through the clay or fine silt. The rush of water through the spaces between the large rocks would be partly stopped by the coarser material, that is, the rock 6 in. in diameter; and the flow through the spaces between the smaller rocks would be partly stopped by the finer material, that is, rock that would pass the 1, 2, 3, or 4-in. mesh. Large quantities of sand may then be deposited on the upper face of the dam. Due to the flow of water under high pressure, this sand would be carried into the dam. The deposit of fine clay or silt on the up-stream face and on top of the dam should produce a practically water-tight structure. Owing to the enormous pressure, water would always seep through the dam, but probably not in quantities sufficient to cause erosion.

When, in 1915, the writer made a trip by boat through 150 miles of the canyon of the Colorado River between Hite, Utah, and Lees Ferry, Ariz., large silt deposits at the mouth of Navajo Creek were observed. This creek joins the Colorado from the south, at a point about 18 miles up stream from the site of the Lees Ferry dam. Large silt deposits also were observed at many other points in the canyon below the mouth of San Juan River. These deposits could be located before they are covered with the back-water, and as the water reached the respective deposits, the material could be loaded on barges by dipper or suction dredgers. It might prove more economical, however, to place rock crushers on the plateau above the dam and transport the crushed material to the dam by chutes. It is thus seen that there is plenty of accessible material for use in the final process of making the rock-fill water-tight.

When the rock-fill becomes water-tight and the reservoir fills, the water may percolate through the 40-ft. layer of sand and boulders. (Fig. 3.) This water would be forced to travel a distance of about 5 700 ft. The indicated

hydraulic gradient is about 1:8, and the writer believes that the frictional resistance to the passage of water through the 40-ft. layer of sand and boulders would be sufficient to consume the head of 700 ft. before the lower toe of the dam is reached, in which case, the velocity of the percolating water near the lower toe of the dam would not erode the foundation. Under these conditions, the dam as designed would be safe against blow-outs caused by the upward hydrostatic pressure of the percolating water on its base.

It is said that the silt-laden waters of the Colorado would be sufficient to tighten the dam. This silt, however, would be deposited at the head of the back-water and not at the dam, which condition favors the completed project. With reservoir full, the back-water would extend up every side canyon and tributary of the Colorado, and would probably extend up the San Juan River, a distance of 80 miles or more. The San Juan, which carries a large quantity of silt would have its own stilling basin and, perhaps, 100 years would pass before the silt would reach the canyon of the Colorado. As each side canyon would have its own stilling basin, the water in the main canyon of the Colorado above the dam would remain practically clear. This is important when it is considered that the water is to be used for the development of power.

As it is proposed to blast about 20 000 000 cu. yd. of rock into the river channel with the first discharge of the mines, arch action may result when this enormous quantity of material falls into the canyon, thus leaving a large open space within the proposed dam. Although such arch action is only a remote possibility, it might be advisable to discharge the mines in both walls of the canyon at the same instant, so that much of the material may meet over the river channel and fall vertically. Perhaps better results can be obtained by discharging the mines in one wall, after those in the other wall, but the writer believes it is impossible to determine just how the rock will fall when the mines are discharged.

Care of Water During Construction.—The writer believes it unnecessary to have the river under control until the dam is raised to an elevation 250 ft. or more above the bed of the river. If the mines were discharged immediately after the flood period has passed, at least eight months would elapse under normal conditions before the river would again be at flood stage. During these eight months, it is expected that the upper face of the dam can be raised 100 ft., or to an elevation of 350 ft. above the bed of the river. The storage capacity of the reservoir between an elevation of 200 and 350 ft. is about 8 000 000 acre-ft.

The writer has described* the flood of 1917, during which the peak at Lees Ferry was about 147 000 sec-ft., one of the highest floods of record originating above that site. An analysis of the records of daily discharge for this flood period shows that the water level in the Lees Ferry Reservoir would have been raised from an elevation of 215 ft. to an elevation of 332 ft. above the bed of the river. The accumulated storage above the 200-ft. level would have been about 7 300 000 acre-ft. and the total volume discharged through the temporary spillway would have been nearly 7 500 000 acre-ft. It is seen, therefore, that in order to prevent such a flood overtopping the unfinished

* *Engineering News-Record*, March 6th, 1919.

dam, it would have been necessary to raise the dam at least 332 ft. above the bed of the river. For the plan of construction herein presented, the writer believes that the most dangerous period would be that during which the dam is being raised from the 200 to the 350-ft. level. During this period, it may be necessary to take care of a summer flood such as that which occurred in 1917.

The water level in the reservoir during construction might be controlled most economically and efficiently by one temporary spillway, at the 200-ft. level, controlled by a Johnson valve having a capacity of 60 000 sec-ft. under a static head of 100 ft. As this spillway is to be used only during construction, it would not be necessary to provide slide-gates so that the Johnson valve may be unwatered for inspection and repairs. If a summer flood, such as that which occurred in 1917, must be taken care of when the dam is nearing the 300-ft. level, the temporary spillway will be taxed to its maximum capacity of 60 000 sec-ft. As the dam is raised above the 300-ft. level, the storage capacity of the reservoir becomes much greater, so that a spillway capacity of less than 60 000 sec-ft. would be required. Under this plan, the dam could be built to its full height without fear of its being overtopped by a flood during construction.

The tunnels leading to the power house may be located at the 600-ft. level. (Fig. 3.) As this paper deals primarily with the design of a rock-fill dam, no attempt will be made to determine the most feasible location for the power tunnels.

The summer floods are caused by the melting snows in Colorado, Utah, and Wyoming. From the records of precipitation, the volume of the coming summer flood can be determined with fair accuracy 60 days in advance. A permanent spillway should be provided at the 600-ft. level so that the water level in the reservoir may be drawn down to provide sufficient capacity to take care of a flood much greater than any heretofore recorded.

Those in charge of the operation of the reservoir would be more concerned with the volume of run-off than with the magnitude of the flood peak. The quantity of snow in the upper basin would indicate whether the expected run-off would be high, low, or average in volume. An extended period of high temperatures, accompanied by a warm rain or "chinook" wind, would cause a flood of great magnitude, but of short duration. It would be impossible to predict such conditions 60 days in advance, but a study of the run-off records for the past 20 years would afford a fairly accurate basis for a prediction as to the time the summer flood may be expected.

Power During Construction.—As coal is abundant within 15 miles from the site, a steam plant could be built to furnish power during construction. After the dam has been raised above the 200-ft. level and the water is passing through the temporary spillway, power may be had by constructing a hydro-electric plant at the outlet of this tunnel.

Conclusions

1.—At the site of the Lees Ferry Dam, the canyon is narrow and the red sandstone is of good quality. These conditions favor the construction of a

rock-fill dam. It seems probable that practically all of the 50 000 000 cu. yd. of rock for the fill can be blasted into place. However, it will be necessary, or at least advisable, to resort to the hand-and-derrick method in placing a part of the rock in the section of the dam above the 600-ft. level. The slopes of the finished dam are not expected to conform exactly with those shown on Fig. 3, but it is the writer's opinion that the average slope should not be steeper.

2.—Owing to the favorable conditions at the site of the dam, the writer believes it entirely feasible to raise the dam to the 250-ft. level with the first discharge of the mines. The quantity of material required to form a dam 250 ft. high depends on the depth to the layer of boulders and the depth to bed-rock. It would be advisable, therefore, to test the river channel for bed-rock by diamond drill borings.

3.—It would not be necessary to by-pass the flood water until the dam has been raised to an elevation of 200 ft. above the bed of the river, except that provision must be made to meet the demand for water for irrigation.

4.—Provision should be made to control the water level in the reservoir above an elevation of 200 ft., in order to facilitate the sluicing of fine material into the rock-fill to make it water-tight.

5.—Flood water should not be permitted to overtop the dam after it has been built higher than 200 ft. above the river bed.

6.—A permanent spillway should be provided at the 600-ft. level.

7.—The raw materials necessary for the construction of a rock-fill dam are conveniently situated.

8.—In this project, it is planned to do on a large scale what has often been successfully accomplished on a smaller scale. The writer believes the most uncertain feature of the project is the work of making the rock-fill water-tight. If the seepage through the dam is sufficient to cause erosion, it would not be very expensive, considering the magnitude of the project, to blast in the canyon walls and produce a 6:1 slope from the crest of the dam to the lower toe.

9.—The writer believes that a rock-fill dam properly constructed would be fully as safe as a concrete dam. To the design herein presented for a rock-fill dam, the unit stresses developed in the foundation and abutment walls are low and need be given minor consideration. This type of dam, however, may be subject to failure due to a blow-out which might be caused by the upward hydrostatic pressure of the water percolating under the foundation. The writer believes that the rock-fill dam described in this paper would be safe against blow-outs or erosion. Should erosion occur, no doubt the trouble could be remedied long before there was any real danger of the dam failing.

Should either the rock-fill or a concrete dam be overtopped by a flood, the damage might be sufficient to cause failure. A flood passing over the top of the rock-fill dam would cause at least a partial failure of the dam. The seriousness of the damage would depend on the magnitude and duration of the flood.

Because the capacity of the reservoir is enormous, it is a comparatively simple matter to provide adequate spillway capacity, and the chances of a dam being over-topped by a flood are remote.

10.—A concrete dam built at the Lees Ferry site to raise the water 700 ft. above the bed of the river would have a volume of about 5 400 000 cu. yd. and would probably cost, including spillway structures, about $50 000 000. It is difficult to estimate the cost of a dam of the rock-fill type, but a liberal estimate indicates that it would be little more than half that of the concrete dam.

DISCUSSION

C. R. F. COUTLEE,* M. AM. SOC. C. E. (by letter).—Rock-fill dams are perhaps the most useful appliances in engineering, now that water power has become such a prominent branch of that science. They have been used in India for a long time, the up-stream face being staunched by depositing, first, stone of one-man size, then macadam size, and, finally, sand. Two parallel rock-fills with an earth center, as used at Panama, is a splendid arrangement.

At Kenora, Ont., Canada, there is a rock-fill dam with sluiceways closed by movable stop-logs. It was built by the late Sir John Kennedy, M. Am. Soc. C. E., 30 years ago, the granite rock being deposited in the rapids without particular care. The up-stream face is neither paved nor staunched, and the leakage has remained about constant. The slope through the dam extends from the water surface above to about 1 ft. higher than the surface of the water below. The head is 18 or 20 ft., the slopes are 1:1, and the width 3 ft. above water is 6 ft.

The Georgian Bay Canal Project, Ottawa River, included many large rock-fills which are described in the report for 1908 of the Public Works Department of Canada. In connection with the Ottawa River storage, several rock-fill dams were built from 1909 to 1912.

At the foot of Quinze Lake, the dam is more than 1 mile long and 45 ft. high in middle channel. The embankment was made from each side and closed in mid-stream with a flow of 5 000 sec-ft. running. Large rocks were carried down stream at first, but they soon came to rest and others jammed against them, until gradually the fill grew up stream. Part of the dam, more than 30 ft. high, was deposited on a swamp. The slopes are 1.5:1, with a width of 10 ft. at 5 ft. above full reservoir.

At Kipawa River, at the north end of the lake, a rock-fill was deposited across a crevice in the granite about 100 ft. wide, through which 1 000 sec-ft. poured in a rapid. At the south end of Kipawa Lake, another rock-fill, 500 ft. long, was deposited on bare rock. No attempt was made to staunch any of these dams. At the north end of Kipawa Lake, the leakage is 300 sec-ft., and at the south end about 200 sec-ft., the maximum head approximating 15 ft.

At the Timiskaming Dam, four concrete sluiceways were scoured under during the spring flood of 1912, and the piers settled down into a sand pocket. Satisfactory repairs were made by filling in the gap with rock. The depth was 25 ft. and the head about 15 ft. Down stream from this dam, a flow of 70 000 sec-ft. scoured out a hard boulder bottom to a depth of 20 ft. below the sill, for a width of 400 ft. This was filled with 30 000 cu. yd. of loose rock, into which the next high flow scoured 15 ft. in depth. Loose rock was again used for filling, and further scouring has not taken place. Descriptions of these works are given in the Ottawa River Storage Reports of the Public Works Department of Canada for 1913 and 1915, which, although out of print, are available in libraries. In constructing the branch of the Canadian

* Ottawa, Ont., Canada.

Pacific Railway from Toronto north to Sudbury, Ont., rock embankments were made to sink into muskeg swamps with considerable certainty as to alignment.

A rock-fill becomes part of the geology of the country, it will not overturn nor slide on its base, and it will stand overtopping for some time. It is very strong, but leaky. To staunch the face, the voids should be gradually reduced with stone decreasing to macadam size and then a covering of earth should be applied at slope of from 3 : 1 to 7 : 1. Brush mattresses are not desirable for staunching, as they bridge voids and eventually break suddenly. At Frank, Alberta, Canada, the great rock slide, 2 miles long and 2 miles wide, occupied the whole bed of the middle fork of the Old Man River. Without gathering much head, the small river, however, has been able to seep through.

A great rock-fill dam might be constructed across the St. Lawrence River, near Rivière du Loup, Que., Canada, requiring 70 000 000 cu. yd., which could be deposited after twin locks, similar to those at Panama, had been built in Hare Island. The sluiceways to pass 800 000 sec-ft. could also be constructed in the dry before any rock was deposited. This would bar out the tide, leaving the water of the upper river fresh, with a current in the outward direction only. There would be no more tidal fluctuation or choking by tide-borne ice, and a 40-ft. depth, without flooding, would be secured. In a similar manner, a rock-fill, 400 or 500 ft. wide on top, could be built across the Hudson River between New York and New Jersey, which would keep the river at a constant level as far as Albany, N. Y.

H. B. Muckleston,* M. Am. Soc. C. E. (by letter).—This paper describes a proposal to dam the Colorado River by creating artificially an enormous landslip from the canyon walls. Except for the artificial feature, the proposal is not without precedent.

The same thing occurred naturally in 1893 on the Birahi-Ganga, a small stream in India. In this case, the dam was about 900 ft. high, 11 000 ft. wide at the base, and 2 000 ft. wide at the top. The catchment area above the dam was small, about 90 sq. miles, so that the discharge cannot have been very great. Nevertheless, the dam went out as soon as it was topped.

In view of the author's proposals for staunching his dam, it is interesting to learn that the Gohna Lake Dam referred to, leaked 350 sec-ft., long before it was topped. A full account of this failure has been published.†

Edwin H. Warner,‡ M. Am. Soc. C. E. (by letter).—The author presents a plan for a rock-fill dam 780 ft. high and greater in magnitude than any dam of which there is a record. The type proposed is archaic, developed when low first cost was a necessity, which necessity does not exist in the case under consideration.

The profile, Fig. 3, is conservative; the notation on it "rock-fill made impervious" is reminiscent of "let there be light and there was light". With this

* Chf. Engr., Lethbridge Northern Irrig. Dist., Lethbridge, Alberta, Canada.

†Records of the Public Works Department of India, No. 324; *Journal*, Soc. of Arts, London, March, 1896; and, also, "Water Power", by the late J. P. Frizell, M. Am. Soc. C. E., p. 250.

‡ Cons. Engr., San Francisco, Calif.

notation in effect, and the author's statement that there shall be no overtopping of the dam by flood waters after the dam is completed, it is not apparent why the toe should be carried 1 mile down stream. The profiles of rock-fill dams illustrated in the "Design and Construction of Dams", by Edward Wegmann, M. Am. Soc. C. E., show no such exaggeration of the down-stream slope.

The author's first step in construction is to drop 1 000 000 cu. yd. into the bed of the stream, making a dam 250 ft. high, "this will settle 60 ft. through sand and silt which will be forced up". The probability that this settlement will occur does not seem great. Much greater, however, is the probability that, as he says, a flood of 100 000 sec-ft. will strew his rock down stream leaving a dam 100 ft. high; the "blessing in disguise" part of the operation in "leaving a solid foundation on which to build" seems somewhat problematical both as to the "blessing" and the "solid foundation". The next step is dropping 20 000 000 cu. yd. in one shot, on top of the first 1 000 000 cu. yd., thus forming "a mud bank immediately up stream" and cutting off the irrigator's supply for "from 10 to 70 days" until the mud bank is overtopped and washed away. The author states that this "might result in serious damage to the crops, and an understanding should be had with the irrigation interests, in order that they may take proper steps to protect the crops". He does not suggest an arrangement; it could take two forms, namely, the purchase of the crops on 400 000 acres at a cost not much in excess of his estimate of the cost of a rock-fill dam, or supplying water from some other source. Neither of these alternatives seems to be quite practicable, so, perhaps, it is best to be content with the possibility that the "bud bank" will not form, or, if it does, it will not make the complete closure suspected of it.

The author's estimate of 10 cents per cubic yard for blasting 50 000 000 cu. yd., seems to be low when viewed from the cost of explosives alone, and his expectation of landing his rock where he wants it, without rehandling, is negatived by his later statement that "it is impossible to determine just how the rock will fall".

The author believes the dam "can be made water-tight", and states that "there will always be seepage through the dam". To secure water-tightness, he assumes that the graded material, later introduced, will find its appointed place in a structure extending 1½ miles along the river. Perhaps it will; who knows?

The author concludes that the dam will cost little more than half that of a concrete dam, the cost of which he places at $50 000 000. Having assigned $5 000 000 as the cost of the rock-fill, it would be of interest to know what items enter into the remaining $20 000 000.

As a final, or, perhaps, more properly, first point in the matter of the author's design and construction methods, there is nothing advanced to satisfy the bond-buyer's engineer, who is obsessed by a curiosity as to what is to be done with the money his people are asked to furnish; he insists on facts and, at times, declares he can furnish his own opinions, which, being his own, satisfy him. That his opinion would be unfavorable to a rock-fill dam, 780 ft. high, is certain, even conceding as does the writer, that water-tightness is not, beyond reasonable limits, an absolute necessity.

His opinion would likewise be unfavorable to a concrete dam of similar height, as the unit stresses in the concrete would exceed the present limits of good practice. The only dam that will meet his approval is a gravity type, the lower section of which will be mammoth rubble laid in cement mortar and carried up to a point where concrete can be safely used. The author's splendid monograph "Colorado River and Its Utilization",* furnishes data which point to a lesser height of dam than that proposed in this paper.

The mean annual flow of the Colorado at Lees Ferry is 15 000 000 acre-ft., but in order to satisfy fully all irrigation needs, 13 000 000 acre-ft. are required. The storage of 25 000 000 acre-ft. will give proper regulation and, with a dam approximately 450 ft. high, 800 000 h. p. can be developed.

The value of property below Lees Ferry makes imperative the adoption of a dam of the gravity type, because it is the only type which will have the approval of the farmer, the bond-buyer's engineer, and the Federal and State regulatory bodies. The writer holds firmly to the opinion that an arch dam can be designed and built, which will be as secure against failure as the gravity dam, but consideration of this type is precluded by the fact that agreement in this opinion is not to be had from the several parties at interest.

If the author finds less constructive criticism in the foregoing than he asks for, he may find contentment in this, that both he and the writer are dealing largely in opinions on a matter which greatly transcends present knowledge; hence, each may hold to his own without undue prejudice.

F. A. Noetzli,† Assoc. M. Am. Soc. C. E. (by letter).—Hydraulic engineers of the West are keenly interested in the deliberations of the Colorado River Commission which, under the leadership of Herbert Hoover, M. Am. Soc. C. E., has accomplished a great deal toward smoothing out the conflicting interests of the various border States interested in the control and utilization of the waters of the Colorado River. In view of the fact that, by providing adequate reservoirs for storing the flood waters of the river at the various excellent reservoir sites on the main river and its tributaries, sufficient water appears to be available to satisfy, in every State concerned, all reasonable claims for irrigation and power for generations to come, it is to be hoped that the Colorado River Commission will soon be successful in initiating the construction of a large dam on this river.

At present, about 14 000 000 acre-ft. of water, out of an average of 16 000 000 acre-ft., is wasted annually into the Gulf of California. This would be enough water to irrigate at least 3 000 000 acres of land in addition to the area already irrigated from the river. According to the author, "more than 4 000 000 continuous horse-power may be developed between Green River and Boulder Canyon." Such figures should convince one that most of the present quibbling, with regard to sacrificing the rights of certain States for the benefit of others, has few engineering facts on which to rely.

Mr. La Rue's excellent paper will be welcomed by many engineers, as it affords an opportunity to discuss the feasibility of a large dam on the Colorado River. The thorough investigation of the whole project, made by the U. S.

* Published as *Water Supply Paper No. 395*, by the U. S. Geological Survey.
† Chf. Engr., Bissell & Sinnicks, San Francisco, Calif.

Reclamation Service, and more particularly the surveys and borings at the Boulder Canyon Dam site, would appear to have furnished sufficient information to warrant the conclusion that the construction of a high dam across the Colorado River is feasible.

The author suggests the construction of a rock-fill dam by blasting down the canyon walls. In order that the discussion might be based on definite dimensions, he chose the Lees Ferry site, without, however, advocating this or any other specific project.

The suggestion of blasting enormous masses of rock from both sides of the canyon walls is fascinating, but it might prove to be a dangerous experiment on a stupendous scale. There exists no precedent on which to rely. All the evidence that may be gathered from past experience with rock-fill dams would indicate an almost certain failure. The writer is unable to agree with the statement of Mr. La Rue, that (in case of a flood of 100 000 sec-ft. passing over the dam to a depth of 8 ft.), "* * * to believe that, after the flood, the unfinished dam would retain a height of 100 ft. or more above the average low-water level, seems more reasonable than that all of it would be carried away. * * *" In the past, whenever water has overtopped a rock-fill dam for 1 ft., or even less, the dam almost invariably was destroyed. The erosion, which may be assumed to have started at the crest, increased the depth and, therefore, the velocity of the flowing water. Since, as the author states, "* * * weight of bodies that can be moved by a current varies as the sixth power of the velocity * * *", the erosive action of the water overtopping a rock-fill dam increases at a tremendous rate, and soon will move also the largest sized rocks, inasmuch as the waters in the reservoir are discharged, in addition to the flood flow that started the failure. Fig. 8 is a view from the new Lower Otay Dam, down the canyon through which the waters passed, that, in 1916, destroyed the former rock-fill dam. The photograph was taken by the writer early in 1922 and clearly shows the lighter colors of the deeper parts of the canyon, the side walls of which, for miles, were stripped of every loose particle, down to smooth solid rock. The force of the water will be appreciated from such a view. From a description of the failure of the Lower Otay Dam,* it appears that the reservoir of 40 000 acre-ft. capacity was emptied in 2½ hours. This corresponds to an average flow of nearly 200 000 sec-ft. for 2½ hours. In addition, the run-off from the water-shed amounted to about 35 000 sec-ft. The maximum flood peak was, therefore, probably in excess of 300 000 sec-ft., and the water rushed down the canyon with an average velocity of about 18 ft. per sec. Referring again to the possibility of the Lees Ferry dam being overtopped during construction to a depth of 8 ft. by an unexpected flood of 100 000 sec-ft., it appears doubtful if more than perhaps a few of the largest blocks of the dam would stay within the area of the foundation.

Assuming, however, that the unfinished dam would retain a height of 100 ft. after the flood, it may be well to consider also the effect of the water that would be released from the reservoir between the 100 and 250-ft. level The reservoir capacity between these two levels is several million acre-feet, and

* U. S. Geological Survey, *Water Supply Paper No. 426*, p. 25.



FIG. 8.—VIEW DOWN CANYON FROM NEW LOWER OTAY DAM.

most of this water would be released during the first day, producing, with the assumed flow of 100 000 sec-ft., a flood peak, below the dam, of probably more than 1 000 000 sec-ft. The effect of such a flow on the Imperial Valley and the bottom-lands along the Lower Colorado River would be disastrous.

However, it might perhaps be possible to devise a program of construction by which the water would be prevented from overtopping the dam. The question is then narrowed down to the problem of making the dam water-tight during construction, as well as afterward.

Mr. La Rue suggests blasting about 20 000 000 cu. yd. of rock into the river channel with the first discharge of mines located on both sides of the canyon. It hardly can be assumed that this large quantity of rock would settle and spread evenly across the whole river, so as to form a more or less level barrier about 250 ft. in height, which would be the object of this first operation. At the 250-ft. level, the canyon is about 600 ft. wide, and, unless a great quantity of excess powder is used, the blasted rock would probably slide down the sides of the canyon, instead of being hurled a distance between 300 and 700 ft., measured horizontally, to the location where the designer of the dam would like it to drop. The danger evidently exists of a continuous low channel being formed approximately along the axis of the canyon through which the water might escape and destroy the dam before the level in the reservoir had reached the spillway tunnel. A rock-fill dam of the dimensions suggested by the author would no doubt be stable if it was possible to make the dam water-tight, or at least so to reduce the velocity of water percolating through the dam that it would not start erosion and blow-outs.

Past experience with earth and rock-fill dams has shown that, in general, for dams of considerable height, this is possible only by lining the up-stream face, or by providing a water-tight membrane, such as a concrete or steel sheet core-wall, or a puddle of clayey material, etc.

The author believes that "with the river under control, the rock-fill can be made water-tight by sluicing fine material into the dam. * * *" The method proposed is ingenious. However, a combination of unforeseen conditions, such as arching of the loose material within the dam, and consequent abrupt and uneven settlements from the weight of the added rock in the building of the dam, might rupture the originally water-tight sand and silt cover and a blow-out might follow with disastrous consequences. The Colorado River silt is remarkably water-tight even in layers of a few inches, if the silt is precipitated from still or slowly moving water. However, if the water attains a velocity of 3 ft. per sec. or more, erosion takes place with great rapidity. Such action might occur if a silt membrane, on the up-stream face of the proposed dam, was ruptured by abrupt settlements within the dam.

Furthermore, it must be considered that, even if it should be possible, by sluicing, to make the dam water-tight for a working head of, say, 200 or 300 ft., the possibility still remains of flow and erosion starting when the water level in the reservoir would rise 600 or 700 ft. above the stream bed. The water pressure at the bottom of the full reservoir would be 43 700 lb. per sq. ft., or

about 300 lb. per sq. in. Such a pressure is considered by conservative engineers as the limit for good concrete in masonry dams, and probably few designers would rely on a layer of wet sand and silt to sustain safely such load.

It would seem, therefore, that the construction of a rock-fill dam on the Colorado River involves a number of rather doubtful features which, during the construction of the dam, would hardly be conducive to sound sleep on the part of the responsible engineer, or the people living below the dam.

In his report to Congress, Arthur P. Davis, Past-President, Am. Soc. C. E., Director of the Reclamation Service, has demonstrated that the construction of a masonry dam on the Colorado River is safe and feasible. Hardly any dangers to life and property on the Lower Colorado River are involved in the construction of a masonry dam, and the final designs may show such a structure less costly than a rock-fill dam.

The problem to be solved, therefore, is at which of the various sites should such a masonry dam be built? Under present conditions, such a dam has to serve three purposes: Flood control, irrigation, and the development of hydro-electric power.

Flood control of the Lower Colorado River is urgently needed. The great snowfall of the winter of 1921-22, over the drainage area of the river, may produce at some time in the future exactly the dangerous conditions of which the inhabitants of the Imperial Valley are afraid and which such a dam would almost completely eliminate.

Even by permitting the upper States all the freedom desired in the irrigation of any or all of their land that may be entitled to it or that is susceptible of irrigation from this source of supply, enough water would still remain to irrigate 1 000 000 or 2 000 000 acres of additional land in Arizona, California, and Mexico.

Between 500 000 and 1 000 000 hydro-electric horse-power may be developed economically as a by-product. This is more than the market may possibly absorb for many years within economical distance of distribution. According to the author's figures there are, in addition, more than 4 000 000 h. p. capable of development between Green River and Boulder Canyon, enough for generations to come. In view of these facts, it is to be hoped that the various States concerned will soon come together for united action and empower the Federal Government to proceed with the construction of a dam on the Colorado River. The embracing of a maximum part of the water-shed to afford the greatest possible protection from floods, the closer proximity to the large area of irrigable lands, and the closeness to the market for power, all favor the building of a dam at the Boulder Canyon site. After this dam is built, possibly the experiment of constructing a rock-fill dam at Lees Ferry may be undertaken, as suggested by the author. If this construction was undertaken when the Boulder Canyon Reservoir was purposely nearly empty, no great damage might occur if the Lees Ferry rock-fill dam should suffer a blow-out during construction.

The writer is particularly interested in the problems of the Colorado River for the reason that he believes he has solved in part what might be called the problem of the Lower Colorado River below Boulder Canyon. To the present

time, it was believed by many engineers who investigated the lower course of the river, that it was not economically feasible to build a high dam across the river for power purposes. The reason for the negative result of these investigations lies probably in the fact that bed-rock, at the deepest point in the river channel, is more than 100 ft. below the present stream bed. Furthermore, the flow in the river is seldom less than 4 000 sec-ft., and floods of 150 000 to 200 000 sec-ft. may be expected almost every year.

About 5 miles above Parker, Ariz., the solid rock banks on both sides of the Colorado rise about 100 to 120 ft. above the river bed and are 600 to 700 ft. apart. Through the protruding rock hills on the Arizona side, it is proposed to cut a channel about 250 ft. wide, somewhat lower than the present river bed, and, by means of a coffer-dam, to divert the flow of the river during the construction of an earth and rock-fill dam across the main river channel. After the main dam has been completed to the crest, and a spillway of 200 000 sec-ft. capacity is excavated at a favorable location on the California side of the river, the temporary diversion channel may be closed by a concrete arch dam, at the foot of which the power-house might be built. The main dam, which would be about 100 ft. high, would be of the earth and rock-fill type with a concrete core-wall reaching to bed-rock or to a double row of sheet-piling to prevent percolation through the dam.

With a load factor of 70% and the quantity of stored water available between the sill of the spillway and the crest of the 15-ft. gates on top of the spillway, about 100 000 h. p. could be developed. The remainder of the reservoir is thereby assumed to be silted up. The intake to the turbines will be through the arch dam, thus affording a minimum cost of installation and a maximum degree of efficiency for operation. At the tail-race of the power plant, a part of the water discharged by the turbines may be taken in a canal and used for the irrigation of about 120 000 acres of land a short distance below the dam. Complete plans for the irrigation of this land were prepared some time ago by the U. S. Indian Service.

The temporary diversion channel would require the excavation of about 1 000 000 cu. yd. of rock, of which a part might be used in the construction of the main dam.

Thus, by the somewhat unusual method of diverting a mighty river into a new channel, then building, without any undue risks, a dam across the main channel, and closing afterward the temporary diversion canyon by a concrete arch dam, under the most favorable conditions as to rock foundation and side abutments, a safe and economical solution, in part, of the problem of the Lower Colorado River is effected.

It is of further interest to note that the proposed reservoir near Parker would afford an excellent opportunity for temporary flood control for Imperial Valley during the construction of the Boulder Canyon Dam. The time required for building the Boulder Canyon Dam may be 10 years, while the dam at Parker could be constructed within less than 2 years. The capacity of the Parker Reservoir is about 1 500 000 acre-ft., and the silt deposits from the river may reduce this capacity by about 100 000 acre-ft. per year. As the dam will

be used later mainly for creating head for the power plant, the silting process of the reservoir is of minor importance.

KIRK BRYAN,* ESQ. (by letter).†—During the extensive surveys of the Colorado River by the U. S. Geological Survey in 1921, the writer was detailed to conduct geological work in the vicinity of Lees Ferry. The results of this work, in so far as they affect the building of a dam at Lees Ferry, are presented herewith. The paper by Mr. La Rue has raised definite questions in the solution of which the geologic facts are pertinent.

Rocks Involved.—The rocks in the vicinity of Lees Ferry are sedimentary in character and form great beds of like material that are persistent in composition over large areas. The character of these rocks is given in Table 2 and their distribution is shown on the geologic map, Fig. 9, from which also the reader will be able to locate various features of the topography which are discussed subsequently. Fig. 10 shows the formations from the Kaibab limestone to the top of the Jurassic sandstone with the characteristic topography of each formation.

TABLE 2.

	General Section of Rocks near Lees Ferry.	Feet.
Jurassic.....	Brown thin bedded sandstones and shales. (Wingate and Navajo of Gregory). Massive tangentially cross-bedded red to buff sandstones. No parting visible at the center, but upper half has lenses of dense gray limestone 6 in. to 3 ft. thick at intervals and near the top, nodules of limonite the size of peas are common. In the Vermilion Cliffs the upper half is distinctly lighter in color.	? 1 100-1 200
Triassic.....	Chinle formation (Upper Triassic). Blue, green, and red shales; white, gray, purple, and red sandstones; and cherty limestones. Upper part of Chinle formation consists of heavy bedded sandstones and red shales. Lower part contains fossil wood.	1 000±
	Shinarump conglomerate (Upper Triassic?). Gray conglomerate with lenses of sandstone and shale; much fossil wood.	0-40
	Unconformity.	
	Moenkopi formation (Lower Triassic). Red sandy shales and thin bedded sandstones with seams of gypsum. In places has beds of red and gray sandstone 2 to 6 ft. thick, and in one locality 12 ft. of gypsiferous limestone. Base, generally, 1 to 10 ft. of chert conglomerate.	500±
	Unconformity.	
Permian.....	Kaibab limestone. Yellow limestone with numerous more or less rounded nodules of chert.	250
	Coconino sandstone. Gray cross bedded massive sandstone.	300
	Unconformity (?) Hermit (?) shale. Red shale and sandstone.	
	Unconformity (?). (Not observed).	
Permian (?) and Pennsylvanian.....	Supai formation. Red shale with beds of blue limestone.	500+ observed

Of these formations, the sandstones of the Jurassic Age form high and conspicuous cliffs in Echo Cliffs, Vermilion Cliffs, and Glen Canyon. In many places also, the almost sheer cliff is continued downward in the ledges of red

* Geologist, U. S. Geological Survey.

† Published by permission of the Director of the U. S. Geological Survey.

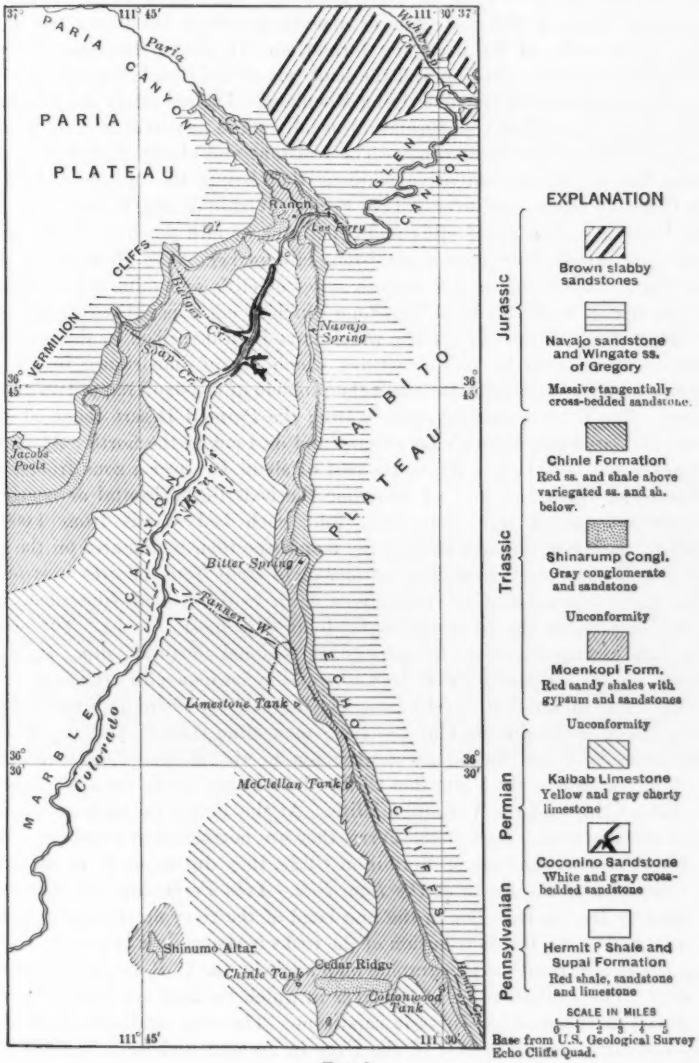

PARIA PLATEAU
PARIA CANYON
Paria R.
GLEN CANYON
Ranch
Lee Ferry
CLIFFS
VERMILION
Badger Cr.
Soap Cr.
Navajo Spring
KAIBITO PLATEAU
Jacobs Pools
MARBLE CANYON
Colorado R.
Bitter Spring
Tanner W.
Limestone Tanks
McClellan Tanks
MOCCASIN
Shinumo Altar
Chinle Tank
Cedar Ridge
Cottonwood Tank
Hermit Shale
111° 45'
111° 30'
36° 45'
36° 30'
EXPLANATION
Jurassic
Brown slabby sandstones
Navajo sandstone and Wingate ss. of Gregory
Massive tangentially cross-bedded sandstone
Triassic
Chinle Formation
Red ss. and shale above variegated ss. and sh. below.
Shinarump Congl.
Gray conglomerate and sandstone
Unconformity
Moenkopi Form.
Red sandy shales with gypsum and sandstones
Unconformity
Permian
Kaibab Limestone
Yellow and gray cherty limestone
Coconino Sandstone
White and gray cross-bedded sandstone
Pennsylvanian
Hermit ? Shale and Supai Formation
Red shale, sandstone and limestone
SCALE IN MILES
0 1 2 3 4 5
Base from U.S. Geological Survey Echo Cliffs Quad.

FIG. 9.

sandstone and intervening shales of the upper part of the Chinle formation. In general, however, the Chinle formation forms ragged badlands or is concealed below the débris from the overlying cliffs. The Shinarump conglomerate commonly forms a cliff and sustains a platform which lies like a fret at the base of the walls of the overlying formation. In places, however, this conglomerate is thin or wanting and the badlands of the Chinle formation merge into the topography of the underlying Moenkopi. The red sandy shale and thin sandstones of the Moenkopi formation usually stand as corrugated cliffs under the protection of the Shinarump. The resistant and cherty Kaibab limestone forms the top of the extensive plateaus that bound the Marble Canyon of the Colorado River southwest of Lees Ferry. Within Marble Canyon, the Kaibab forms a succession of cliffs and often a sheer cliff down to the Coconino sandstone, which is in turn a persistent cliff maker. The Hermit (?) shale and Supai formation together form a series of cliffs and talus slopes down to the great cliff to the Redwall limestone which is not exposed within the area.

General Structure.—The rocks which have been described in the preceding section are affected by two structures which are of importance in the distribution of the rocks and also affect the problem of constructing a dam at Lees Ferry. The Echo monocline extends from the Painted Desert about 50 miles south of Lees Ferry through Lees Ferry and dies out to the north. Along this structure, the beds have a dip to the east of about 10° over a zone from 1 to 5 miles wide. East and west of this zone the beds lie horizontal or nearly so. In consequence of this monoclinal fold, each bed stands from 1 000 to 2 000 ft. higher in the area west of the monocline than in the area on the east. Erosion has proceeded in such a manner as to strip from the area west of the Echo monocline most of the formations down to the Kaibab limestone. The Echo Cliffs mark the zone where the folding is most intense. The second structure of importance is the general or regional northerly dip. In consequence of this structure, the Echo Cliffs and Echo monocline fade out to the north. The inclination of the beds due to this structure is from 1 to 2° and is more noticeable west of the Echo monocline than east of it. Erosion has stripped all the formations to the top of the Kaibab limestone as far north as the base of the Vermilion Cliffs. Lees Ferry lies in the angle between the Echo Cliffs and the Vermilion Cliffs. At the Ferry, the beds are dipping about 10° eastward. East of the Ferry, they are horizontal or nearly so. West of Lees Ferry at the base of Vermilion Cliffs, they dip about 1° to the north.

Character of the Rocks at the Dam Site.—Lees Ferry dam site, as recommended by Mr. La Rue, lies in the first bend of the river up stream from Lees Ferry. In Fig. 11, the distribution of the rocks is shown on a larger scale. The map is redrawn from the topographic map made by the U. S. Geological Survey in 1921. The straight reach of the river in which the dam site is to be located is carved in the massive Jurassic sandstone. The only available site for the power plant is, however, west of the ferry in an area underlain by the Chinle formation. The Echo monocline crosses the area at about the center line of the first loop of the river. West of the lines showing the Echo monocline fracture system in Fig. 11, the rocks dip about 10° to the east. East of these lines they appear to be horizontal or nearly so. Construction, therefore, as far as



FIG. 10.—VIEW DOWN MARBLE CANYON, FROM NEAR LEES FERRY, SHOWING THE VERMILION CLIFFS AND TOPOGRAPHIC EXPRESSION OF FORMATIONS DISCUSSED IN TEXT.

the dam itself is concerned, has to deal only with the massive Jurassic sandstone. This sandstone is unusually massive, having few joints and relatively few bedding planes. Almost all the sand grains are of quartz, imperfectly rounded, but without sharp edges. According to Gregory,* the Navajo sandstone or upper member of the Jurassic sandstone as here treated, consists of sand grains of two sizes. Grains of 0.15 to 0.25 mm. make up 90% of the rock, other grains, averaging 0.65 mm. in diameter, form an interrupted coat on the cross-bedding laminæ. The sandstone is somewhat loosely cemented with calcium carbonate. In addition to the lime cement, the sandstone con-

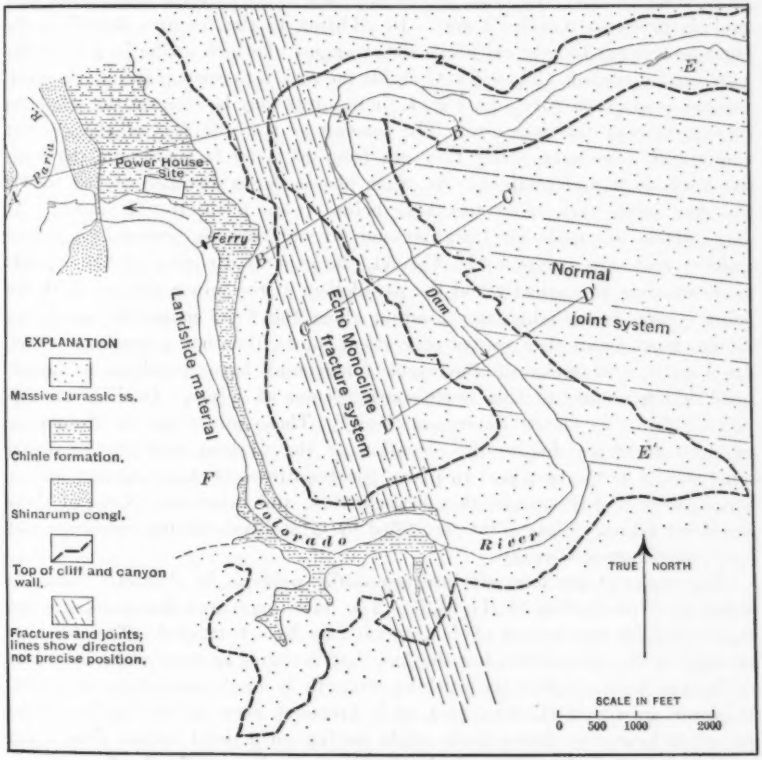


FIG. 11.

tains iron oxides which give it a red or buff color and in places, especially near the top of the formation, these oxides are concentrated in nodules or concretions about the size of peas. In places in the Navajo country, the cement is silicious. At intervals through the sandstone are beds of limestone from 6 to 18 in. in thickness, and these beds fade out laterally in short distances. The

* "Geology of the Navajo Country", U. S. Geological Survey, *Professional Paper 93*, 1917, p. 58.

limestone is gray and crystalline and, as far as known, unfossiliferous. The most striking characteristic of the massive sandstone is its cross-bedded character. The cross-laminæ are from $\frac{1}{8}$ to $\frac{1}{2}$ in. thick and consist of sand grains of two sizes, the finer grains which make up the bulk of each lamina being separated from the next lamina by a layer of larger grains. These cross-laminations extend across the face of the rock in groups of great curved lines that are generally tangent to the curve of lower groups or to the bedding planes. The cross-lamination is on a grand scale, sweeping curves being from 10 to 200 ft., or more, in length. The sandstone in places has a tendency to split parallel to these cross-laminæ. In other places, the rock is so uniform in strength that erosion produces rounded forms. In addition to the features described, the sandstone is cut by vertical joints the strongest of which strike in a generally northwest direction. These joints divide the rock into great plates along which erosion is most effective. In Fig. 4, these joints may be seen on the left hand or east side of the dam site. The spacing of the joints is of the greatest importance. In some places they are from 20 to 50 ft. apart and the walls are very sheer and straight. At other places, for a distance of 100 to 200 ft., the joints are close together, being from 1 to 10 ft. apart. At these places the walls are irregular and are cut back by narrow, steep-sided gulches and side canyons. In Fig. 11, the general direction of these joints is shown diagrammatically and the parallelism of the minor canyons with the joint system is brought out. Folding along the Echo monocline impressed on the rocks is another system of vertical joints, that has a northerly trend. In Fig. 4, the rocks of the right foreground have a schistose appearance which is due to this north-south system of joints. In Fig. 11, this belt of joints is shown diagrammatically. These joints are in the nature of fault fractures due to the elevation of the western part of the region with respect to the eastern. In places the fractures are close together and in other places widely spaced; they branch, fork, and interlace. Some of them are small faults. These joints are filled by crystalline calcium carbonate and iron and manganese oxides.

The rocks of the dam site are essentially uniform in character from the water level to the top of the cliffs. The advantages and disadvantages are thus equal for any height of dam which may be constructed. The crushing strength of the rock is low, but the fact that it stands in great walls in a state of Nature indicates that its crushing strength is ample for a high structure. However, in a rock-fill dam, such as is proposed, some of the blocks will be forced to bear very heavy loads while resting on a point rather than a flat surface. Such rocks will undoubtedly be crushed, and Mr. La Rue is correct in assuming that there will be a large amount of adjustment to load in a rock-fill dam composed of this material. This crushing and adjustment, however, will tend to compact the structure. When such adjustment is taking place, streams of water will doubtless carry away the fine débris and tend to prevent the closing of cavities.

According to Mr. La Rue's proposition, it is expected to blast down the material from the cliffs to form the dam. The rock, however, is so unusual a type that a number of problems will arise in this connection. The writer

with the assistance of Mr. Frank Mitchell made one test shot. A small amount of blasting was also done in the installation of a cable for stream gauging. This limited experience indicates that the massive but porous sandstone presents special problems in blasting. The quantity of powder necessary and the best type of holes or galleries for loading this powder will have to be determined experimentally. It should also be pointed out that the east side of the dam site is affected by vertical joints which strike diagonally to the cliff face. These joints will have a large but unpredictable effect on the blasting. The west face of the dam site from the center line south is similar to the east side, but from the center line north the rock is affected by the north-south system of joints referred to previously. The blasting in this locality may be no more difficult than in other places, but the action will be somewhat different and probably the chunks obtained will be smaller. It seems necessary, therefore, to recommend that preliminary work be done on a rather large scale to determine what the exact conditions of blasting will be, before the first great blast by which Mr. La Rue proposes to raise the dam 250 ft. above river level. When these experiments are conducted it will be possible to obtain data on two important questions. One of these is the size of blocks which can be obtained by blasting and the other is the quantity of fine material which will be produced. It seems obvious that it will be necessary to have very large blocks on the down-stream side of this proposed dam during the period when it is not yet water-tight. An unknown quantity of fine material will be produced during blasting and by the fall of blocks. In Nature, very large blocks can fall the full distance from the top of the cliffs to the bottom without breaking, but the fact that such blocks are found does not indicate how many similar blocks have been crushed by this same fall.

Location of Tunnels.—Dependent on the detailed plans which may be adopted, tunnels may need to be driven in the line of *E E'* and *A A'*, as shown in Fig. 11. Tunnels along the line, *E E'*, either for diversion of the river during construction or as a permanent wasteway after construction will be driven through the massive Jurassic sandstone entirely. Large tunnels can be constructed at a minimum of expense because of the ease of drilling the rock and because its massive character will require no support for the roof. However, unless the velocities in the tunnel are low, a concrete lining will be necessary to prevent wear. It seems likely also that there will be losses to the adjacent porous sandstone if the water in the tunnel is under great pressure and that this water percolating through the sandstone may eventually find or work out channels large enough to produce serious losses from the tunnel and direct the water toward inconvenient places. At Locality *A A'*, a tunnel may be required at a low elevation or at a high elevation to act as a spillway. A tunnel or tunnels to carry water to the power-house will certainly be built here because the flat, east of the present ferry, is the best if not the only available power-house site. The geologic section, Fig. 12, shows the character of rock which will be encountered in tunnels at various elevations. The folding along the monocline brings up the lower formations to river level and above on the west flank of the ridge. Tunnels at low elevations will pass from the massive Jurassic sandstone on the east into the red sandstones and shales of the upper part of

the Chinle formation and thence into the cherty limestone and soft shales of the middle of this formation. Tunnel construction in the middle of the Chinle formation will require thorough support for the roof and a permanent concrete lining. The red sandstones and shale of the upper part of the formation will probably present no difficulties during construction, but a lining will be required as in the Jurassic sandstone. The location of tunnels at the lower elevation becomes then an engineering problem as to relative cost of a short tunnel at Locality *A A'* or a longer tunnel at Locality *E E.'* Tunnels at Locality *A A'* and at an elevation of 650 ft. will be required for carrying water to the power plant. These tunnels will be wholly within the Jurassic sandstone and require no comment either on their construction or location.

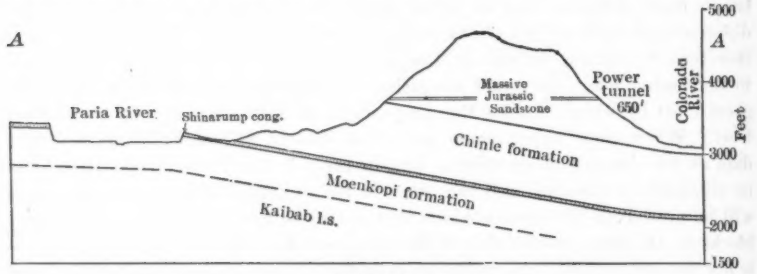


FIG. 12.

Leakage.—On the assumption that the water-tight dam to an elevation of 720 ft. is constructed in the straight stretch of river east of Lees Ferry, consideration was given to the problem of leakage. The west wall of the reservoir is relatively narrow, but the massive Jurassic sandstone is cut by numerous sealed joints, as mentioned previously. It is considered that these joints will form fairly perfect obstacles to percolation. Similarly, the folding which has carried the Chinle formation up to elevations of about 3 500 ft. on the west side of the ridge near the present ferry has laid a dam of rather tight rocks in the path of percolating waters. On the east side of the dam the reservoir wall is formed by the massive Jurassic sandstone. The minimum distance through this wall is about 6 000 ft. A certain amount of direct percolation through the pores of the rock may be expected, but most of the leakage will occur along vertical joints. However, the direction of these joints, as shown on Fig. 11, is nearly at right angles to the direction of this flow. If considerable leakage should take place through the sandstone and large springs break out below the toe of the dam and in the canyon at Locality *E'*, it seems unlikely that such leaks would affect the stability of the canyon wall.

Depth of the River Channel.—During mean high water the depth at the gauging station above Lees Ferry ranges from 13 to 17 ft. The bed of the river is composed largely of sand. Near the margin of the channel are numerous large rocks which have fallen from the cliffs. Many of these seem to be relatively stable in position and not easily dislodged by floods.

It is practically impossible to predict the distance to bed-rock in such a channel. If the river has cut its bed normally, the thickness of gravel and

sand will be only that of scour and fill during great floods. Such scour and fill should be relatively moderate in amount, but is an unpredictable quantity. The Shinarump conglomerate crosses the river channel west of the ferry (see Fig. 11). Mr. La Rue believes that bed-rock extends across the channel at this point at low water. If this ledge exists, then there has been no general silting of the channel. At the dam site there may be, however, a hole or deep in the rock bed of the river due to the presence of a fossil plunge-pool. A plunge-pool or over-deepening of the stream bed occurs at all falls and it seems possible, although there is no direct evidence, that the Glen Canyon of Colorado River was cut back as a head-water falls from the line of Echo Cliffs eastward. As the falls receded, successive plunge-pools would be filled with gravel to bring the river to a uniform grade. It seems necessary for the determination of the depth to bed-rock to proceed in the usual manner by drilling.

Observations on the Scheme for a Rock-Fill Dam.—Mr. La Rue's scheme for a rock-fill dam, which is to be made water-tight by sluicing in a fine material during construction, is a bold solution of the problem for a dam at Lees Ferry. The difficulty of obtaining suitable material as well as the isolation of the locality required such a solution, and the author is to be complimented on his originality and vision. In advance of building such a dam it may be assumed that the thickness and character of the sand and gravel above bed-rock will have been determined and that experiments will have been conducted to determine the best method of blasting the massive sandstone walls.

According to Mr. La Rue, fine material for filling the interstices of the dam can be found up stream at various sand-bars and banks. A roadway up the canyon is impracticable and the navigation of the canyon is attended with some difficulties, even with the assumption that preliminary blasting will raise the water level and pond the water for a distance of 15 or 20 miles. Therefore, another source of fine material was sought. The Moenkopi formation consists largely of sandy shale and might make suitable material for filling such a dam except for its rather high content of gypsum and other soluble salts. Continual solution of this material would make it an ineffective filler for small spaces. Similarly, the base of the Chinle formation, consisting of gray, green, and blue shales, is suitable in that it is clayey, particularly where it is weathered, but this formation also contains a considerable quantity of soluble material. Systematic sampling and some simple laboratory work might show that these clays could be used. The upper part of the Chinle formation consists of heavy, bedded, earthy sandstone and sandy shales, and this part of the formation appears to be rather free from salts, but it is so exposed in cliffs that it can not be easily quarried. It occurred to the writer, therefore, that the heterogeneous landslide material on the south bank of Colorado River opposite the ferry, in the general locality marked *F* in Fig. 11, might be a suitable source for part of this material. The material has been in position for some time and the soluble matter has doubtless been leached out. There is also a large sand dune which is free of soluble matter. By installing a crusher and mixer, the surface of this slump or landslide area and the dune could be stripped with steam shovels and crushed, obtaining as a result a mixture of disintegrated

shale and leached sandstone which would have a texture similar to loam. Loaded into cars, this material could be brought across the river and carried to the top of the bluff on an inclined railway. The use of this material depends on its relative cost.

It seems probable that in blasting the cliffs a large quantity of fine material would be obtained and, unless precautions are taken, it is likely to be lost. If as a preliminary to the large blast, which is to raise the dam to 250 ft., a small low dam is thrown across the river at or slightly below the toe, the dam would then be built in slack water and much more fine material would be saved.

Mr. La Rue's scheme is generalized, and he has assumed that blasting in the cliffs would produce a relatively even crest line at any given point on the dam, but it should be borne in mind that the material will slide into the dam site in the manner of a series of talus cones and the low point of the dam at any cross-section will be near the center line of the river. The difficulties may be visualized by a consideration of the cross-sections shown in Fig. 13. Line *C C'* is at the center of the dam as proposed by Mr. La Rue, and the other lines at the upper and lower points, respectively. In Cross-section *C C'*, a 1 on 2 slope has been projected. It is possible that when blasted the material will slide into the lower part of the canyon on this slope, but it is unlikely that the heap of loose material will stand on this slope. It seems more likely that it will assume a slope between 35 and 50 degrees. The existing talus which occurs at intervals along the canyon walls doubtless has the minimum slope. Three talus cones shown on the large topographic map of the dam site have maximum slopes between 29° 30′ and 33° 30′. The artificial cones will be less stable and will probably have coarser material. Overtopping of the dam during construction is a serious matter if the water will be concentrated in a V-shaped trough near the center line of the dam, because the maximum velocities will thereby be obtained. It may be necessary, therefore, to hand-place a considerable part of the material near the longitudinal center line of the dam, or to blast in an excess of material so that the center line will be the required height.

Materials for Construction.—The local materials available for general construction purposes are considered in the following paragraphs.

The Coconino sandstone, Hermit (?) shale, and Supai formation are exposed only in the Marble Canyon or its tributary canyons. The outcrops are so inaccessible that these formations may be dismissed as sources of material.

The Kaibab limestone is exposed over large areas southwest of Lees Ferry, and there are numerous places where quarrying can be done economically. The formation is about 250 ft. thick and consists almost exclusively of yellow limestone with nodules of chert. No beds were seen which gave any promise of being valuable for making cement. Dimension stone of large or small size can be readily obtained. The hard chert nodules are so numerous that the cost of facing blocks with a chisel or hammer will be large. The rock can be considered as available only for rough ashlar or rubble masonry. It would, however, make excellent crushed rock for concrete or macadam.

The Moenkopi formation is a sandy shale with interbedded thin sandstones and contains considerable gypsum. It is valueless for structural purposes.

The Shinarump conglomerate is about 40 ft. thick and forms a gray ledge above the "dug way" and the ridge on the east side of Paria Creek. It is a cross-bedded conglomerate of well-rounded silicious pebbles with lenses of sandstone and small lenses of green shale. Fragments of fossil wood and logs of trees are abundant. This conglomerate is hard and durable, has widely

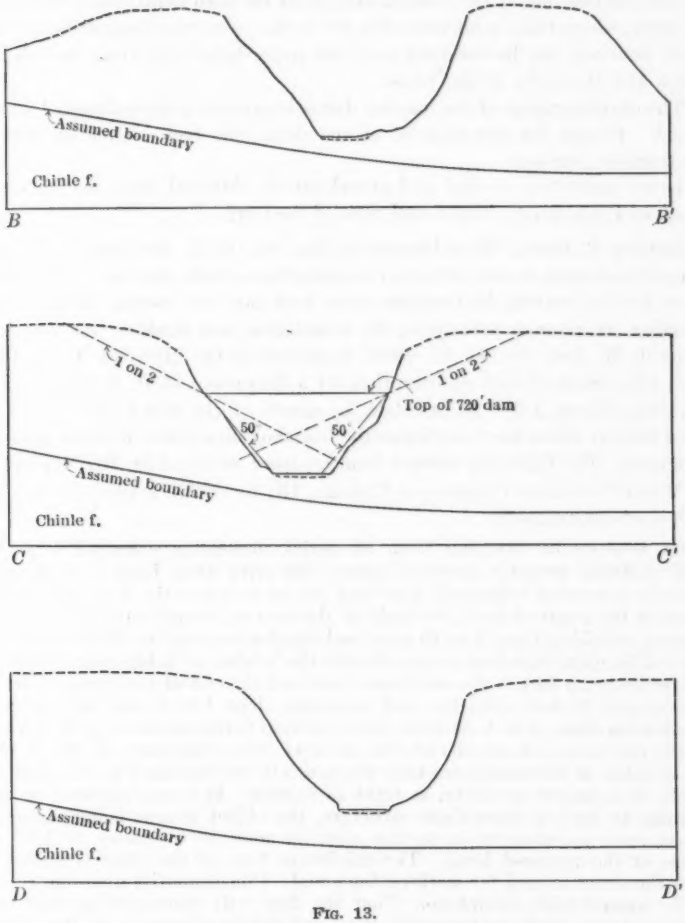


FIG. 13.

spaced vertical joints, and is an excellent rock for rubble or Cyclopean masonry. It is somewhat too hard and coarse-grained to be easily worked into blocks of uniform dimension.

The Chinle formation is a very complex unit consisting of gray, green, and blue shale below, white and purplish sandstone and shales, cherty limestone and

shales in the middle part, and in the upper part massive red sandstones and shales. Only the upper part contains material of any value for construction purposes. The red sandstones have a pleasing color, can be obtained in large and small sizes, and are easily worked. They have no great crushing strength, nor are they suitable for masonry which is to be alternately wet and dry. The best site for a quarry from which to obtain these sandstones that might be used for the construction of the power-house, is on the high point south of the present ferry, where these sandstones dip 10° to the east. Small quantities of such stone, however, can be obtained near the power-house site from the adjacent ledges and the rocks of the talus.

The characteristics of the massive Jurassic sandstone have already been discussed. Except for the rock-fill of the dam, this rock is not suitable for construction purposes.

Large quantities of sand and gravel can be obtained from the bar at the mouth of Paria River about a mile west of the ferry.

ARTHUR P. DAVIS,* PAST-PRESIDENT, AM. SOC. C. E. (by letter).—The first proposition known to the writer for constructing a high dam in a large stream on an alluvial bottom, by dumping loose rock into the flowing water without diverting the river or excavating the foundation, was made by the late A. G. Menocal, M. Am. Soc. C. E., Chief Engineer of the Maritime Canal Company, who proposed such an operation for a dam about 65 ft. in height on the San Juan River, a few miles below the mouth of the San Carlos at a point called Ochoa, where the river forms the boundary line between Costa Rica and Nicaragua. The following extract from a paper† prepared by Mr. Menocal for the Water Commerce Congress at Chicago, Ill., in 1893, describes the dam and method of construction:

"It consists in dumping from an aerial suspension conveyor large and small material properly assorted, across the river from bank to bank until a barrier is created sufficiently high and strong to arrest the flow and hold the waters at the required level, the body of the dam to be made up of large blocks of stone, weighing from 1 to 10 tons, and smaller material to fill the voids. Its base will be quite broad as compared with the height, probably from 400 to 500 feet between the foot of the upstream slope and the end of the apron. The top is estimated 30 feet wide, the rock upstream slope 1 to 1, and the apron, or downstream slope, 4 to 1, with the lower portion flattening down to 5 or 6 to 1. On the upstream side small material, such as stone, fragments of gravel, clay, etc., selected as circumstances may require, will be deposited as the work advances, in sufficient quantity, as tight as wanted. It is not expected or even desirable to have a water-tight structure, the object sought being simply to oppose such an obstruction to the river as may be necessary to hold the waters at the required level. The minimum flow of the river is about ten times the water needed for working the canal. Consequently, nine-tenths of it can be wasted with advantage. That the dam will eventually become tight there can be no doubt, as the small drifts and detritus forced in by the current will gradually fill the voids and consolidate the structure.

"The method of construction will be quite simple. After protecting the abutments against possible erosion, large pieces of rock will be dumped in the bed of the stream from three or four cableways spanning the valley. The

* Director, U. S. Reclamation Service, Washington, D. C.

† H. R. Document No. 279, 54th Cong., 1st Session, p. 53.

material should be distributed uniformly over the area under the main portion of the dam, commencing up stream and keeping up, as nearly as possible, an even level. Scouring will soon cause settling of the blocks into firmer soil, the upper level in the meantime being constantly raised by depositing more stone, while the small material is being forced by the current into the voids, and the overflow dislodging and rearranging the unstable blocks until they reach a final resting place. This process to be continued until the resistance at the bottom becomes so great as to check scouring due to maximum pressure, when the dam will be carried up to the desired level. The river, in the meantime running over the mound, will readjust the material in and adapt the apron to the necessary conditions of stability to withstand the effect of the fall, and carry off the water safely. If the dam is then raised so as to shut off a whole or the largest part of the river flow, which can by that time be discharged over the waste weirs, the structure will be permanent."

It was proposed that this dam be an overflow weir, and this feature has been generally condemned.

The Nicaragua Canal Board, consisting of the late William Ludlow, M. Am. Soc. C. E., Mordecai T. Endicott, Past-President, Am. Soc. C. E., and the late Alfred Noble, Past-President, Am. Soc. C. E., studied the plans of the company and made the following comments* on this design:

"It appears from this account of typical existing dams that, although rock-fill dams are not new, and although weirs have been built on sand and maintained successfully, the Ochoa Dam is actually without precedent in its more serious aspects, and its construction will be far more difficult than any that have been mentioned. The successful rock-fill dams have impervious foundations, and are made water-tight by sheathing or masonry. The materials in them are in great part hand laid, adding greatly to their stability, and none of them has been built in conflict with the forces of a great river or was intended to be used as a weir for the discharge of floods.

"As to this last point, the Board is clear in its judgment that no such endeavor should be made in the case of the Ochoa Dam, as involving an unwarrantable hazard to the safety of the structure and of the canal navigation."

* * * * * * * *

"The weakest point of a dam built as the company proposes would probably be at the river bank, where the loose rock will meet the steep clay slope. The method proposed for protecting this point by lines of sheet piling and concrete core seems entirely inadequate. A better construction would involve the use of a caisson on each side of the river, located so as to be partly in the river channel and partly in the firm clay bank, and sunk by the pneumatic process to bed rock if within reach by that means, or if not at least below reach of scour. It seems probable that either bed rock or a mass of boulders would be found near sea level, elevation 0. The caisson should be surmounted by a concrete wall, extending up the hillside beyond the caisson to the highest elevation to be reached by the water surface after the completion of the dam, say 114 or 116. A trench 20 feet or so in depth, or deeper if necessary to reach firm material, should be excavated in the hillside for the core wall. This work done, the river banks and hillsides below high-water level should be covered with a heavy mass of large stones and an additional amount held in reserve before depositing any rock in the river. The result of this would be to contract somewhat the width of the river channel, which would at once scour out so as to maintain its normal cross section. A considerable scour would be desirable, because it is important that the rock mass be sunk deeply

* H. R. Document No. 279, 54th Cong., 1st Session, pp. 61–64.

into the river bed, since, if rock or a bed of boulders or heavy gravel can be reached, the safety of the structure will be increased greatly. In depositing rock in the river those methods should be used which will induce scour, and possibly the building out from each side may best answer this purpose."

* * * * * * * *

"The rock mound would be quite permeable, and the water pressure against it would vary with the volume of the discharge. To make it hold water, an embankment, which would form the real dam, the rock mound serving as its support, must be built on the upstream side. It is proposed to make the embankment of fine stone, gravel, clay, etc., with the view that a quantity of these materials should be carried well into the rock mound. An embankment composed largely of clay dumped in the water would take a much flatter slope than 3 : 1. For these reasons the quantity estimated by the company would have to be increased largely, and means may have to be adopted, not contemplated by the company, to render the dam tight enough to answer its purpose, as the permissible leakage is much less than the company has assumed.

"By the addition of the embankment, and the retardation by friction of the flow of water through the underlying sand bed, the dam will be rendered more secure from undercutting and consequent settlement; but some doubt as to ultimate security will still remain. Confidence would be gained to a certain extent during the construction of the dam, if no breaches were caused by the annual floods, but it would require a considerable period of time after the completion of the dam to allay all apprehension. If a site can be found where the dam can be built above a rock or other stable foundation, the permanence of the structure will be rendered much more probable and the uncertainties of construction reduced.

"If such a dam becomes breached with the great volume of water standing at a high level against it, destruction would follow quickly. The crest, therefore, should be raised so far above the surface of the water in the pool that a considerable settlement could be sustained without sinking the crest below the water surface."

It should be noted that the Board places emphasis on the importance of scour in settling the foundation as far into the sand as possible. It is also to be noted that this eminent board of engineers was doubtful of the permanent safety of a dam even of this height under these conditions.

The practical application of the method of damming a large river by dumping rock on a bed of silt while the river flowed over it was made by the Southern Pacific Company in 1906, in the spectacular control of the Colorado River when it was flowing into Salton Basin. The same method was used in constructing a coffer-dam across the Colorado River in making the final closure for the Laguna Dam. In neither case, however, was the water allowed to flow permanently over the rock-fill, as proposed by Mr. Menocal, and the rock used was settled deep into the silt by the erosion of the water during construction.

The author proposes no means for effecting a water-tight junction between the rock-fill and its abutments, and the writer sees no effective method of accomplishing such a junction between the proposed silt filling and the rock. Under such a head, water would follow the rock with a velocity likely to become erosive at many points, and thus open avenues of danger. The best solution seems to be a concrete pavement cemented to the abutments with all the

care and precaution that the highest skill can suggest. Cracks which might appear should be sealed with a cement gun at the first opportunity.

In considering the construction of a loose rock dam in Boulder Canyon during the past few years, the Government engineers and those whom they have consulted have regarded the settlement of the loose rock into the soft material as deeply as possible as one of the most important and difficult operations to accomplish, and it seems obvious that it cannot be made perfect without excavating the loose material, but might perhaps with many precautions be made to sufficient extent to secure safety.

The author's proposal to deposit enough rock in the river at one blast to bring the dam to the 250-ft. level seems especially designed to prevent any water action in the direction of scour and of sinking the rock into the silt. He says:

"If the canyon walls were blasted into the river, the heavy mass of material would settle to the level of the boulders, assumed to be 60 ft. below the bed of the river, and some of the fine sand and silt would be forced up."

No reason is given for this important conclusion and the methods proposed seem likely to prevent this desirable result.

If this dam was built by the methods suggested by the author, there would be a considerable part of the structure below the river bed founded on sand and silt. As the water rose in the reservoir after the dam was completed, the head on this silt would gradually increase and no assurance could be given that a critical point might not be reached when the reservoir is nearly full, and sufficient percolation through the foundation might take place to start erosion and cut a channel through the silt. A free passage ever so small, resulting from the slight arching of the rock, that would conduct water through the sand at a velocity due to more than 700 ft. of head, would scour rapidly, carrying considerable rock, until a channel was excavated large enough to allow the rock above to settle, and unless this was accomplished in one motion the scour would continue until the main rock-fill fell into place. If it was arched sufficiently it would not do so promptly, and the scour would continue until the opening would carry a flood much larger than normal and do great damage below. It seems inevitable, however, that eventually the rock-fill would fall in place and stop this. If, in so doing, it should fall below the level of the water in the reservoir, the dam would be overtopped. No loose rock dam has ever been overtopped without failing, and some have failed. None of them has had a 780-ft. head nor 50 000 000 acre-ft. of stored water behind it to complete its destruction.

It should be remembered that the proposed reservoir of 50 000 000 acre-ft. has an area of 235 000 acres, and a discharge of 1 000 000 acre-ft. would lower the reservoir only about 6 ft. This would afford an average flow of 500 000 sec-ft. for 24 hours. No one can doubt that this would destroy the dam faster than it would lower the reservoir. The writer will not attempt to describe the results of suddenly releasing 50 000 000 acre-ft. of water. It would at least submerge and destroy the great Imperial Valley and all improvements along the Lower Colorado River, and doubtless many lives would be lost.

It is readily seen that both the danger of failure and the magnitude of the disaster are greatly augmented with the height of the dam. This law applies especially to an unprecedented or doubtful design.

Where the necessary structure so far exceeds precedent, to build it any higher than needed would be inexcusable. The author appears to consider a capacity of 20 000 000 acre-ft. to be sufficient to regulate the river. The writer agrees with this estimate, but would provide 20% more to store silt and prolong the usefulness of the reservoir. There can be no reason therefore for building a reservoir of greater capacity than about 24 000 000 acre-ft., except for the increased head for power development. This site is so remote from markets and so much farther than many other available sites that there is at present no excuse, and for many years or decades there will be no justification, for developing power at this point. The available head can be utilized by building dams of moderate dimensions at other points without such unprecedented and hazardous characteristics.

The author states that the construction of a dam near Lees Ferry would remove the menace from floods on the lower river, except floods from the Gila River. That statement is erroneous, as the proposed dam would leave about 50 000 sq. miles of drainage area unregulated, besides that of the Gila. It is perfectly proper, as the author does, to except the Gila River, which for brief periods sometimes furnishes floods as great as those of the Colorado; but the area of 50 000 sq. miles between the mouth of the Gila and Lees Ferry should also be excepted. Most of this area can be intercepted by a dam at Boulder Canyon, which is the lowest point in the basin where this can be done, and is accordingly the proper place for storage purposes if the floods are to be controlled.

Table 1 shows a capacity for Lees Ferry Dam of 50 000 000 acre-ft. and for Boulder Canyon Dam of 31 680 000 acre-ft. The results of the actual surveys were probably not available to the author when he submitted his paper. These surveys have been in progress for a long time, and the results are now available and are shown roughly in Table 3 which gives the comparative area and capacity for each given height of the dam, as high as the surveys have been carried at Boulder Canyon.

Table 3 shows that, for the heights given, the Glen Canyon Reservoir would have less capacity than Boulder Canyon.

A dam 780 ft. in height in Glen Canyon near Lees Ferry would introduce not only unnecessary hazards, but would waste a large quantity of water by evaporation. A reservoir of 24 000 000 acre-ft. capacity in Glen Canyon, would have an area of about 125 000 acres, according to the available surveys, whereas one of 50 000 000 acre-ft. capacity would have a surface area of 235 000 acres and unnecessarily expose to evaporation 110 000 acres. This would be at its highest stages during the hot months of June, July, August, and September. Assuming an average excess area of 100 000 acres, and an annual evaporation of 6 ft. from the extra area, 600 000 acre-ft. of water would be wasted annually, all of which would be lost for use at the numerous high dams eventually to be built in the canyon below.

The absence of a precedent to the proposed plans leaves the writer in doubts much the same as those which remained in the minds of the members of the Nicaragua Canal Board previously quoted, and there apparently is sufficient doubt so that it would be inexcusable to attempt such a structure

TABLE 3.

Height of dam above river bed, in feet.	Boulder Canyon Reservoir.		Glen Canyon Reservoir.	
	Area, in acres.	Capacity, in acre-feet.	Area, in acres.	Capacity, in acre-feet.
50	3 500	88 000	4 000	100 000
100	7 500	362 000	7 000	375 000
150	15 500	937 000	10 800	820 000
200	25 000	1 950 000	16 600	1 505 000
250	33 500	3 413 000	24 000	2 520 000
300	44 000	5 350 000	34 000	3 970 000
350	57 000	7 875 000	47 000	5 995 000
400	73 000	11 125 000	63 000	8 745 000
450	90 000	15 200 000	85 000	12 445 000
500	109 000	20 175 000	114 000	17 420 000
550	132 000	26 200 000	145 000	23 895 000
600	157 000	33 425 000	178 000	31 970 000

where more conservative designs are possible or to build the dam under these conditions any higher than necessary.

Opinions concerning the safety of the proposed design can hardly be considered of much value until something is known of the underground conditions, which have not been investigated at Glen Canyon. Still less is it possible, under these conditions, to make intelligent estimates of cost of any type of dam. These conditions at Boulder Canyon have been carefully investigated, and comparative estimates of high concrete and rock-fill dams have been made by men with wide experience in the construction of high dams. These estimates lead to the conclusion that the rock-fill type, with proper precautions for safety, could not be relied on to be much cheaper than a massive concrete structure the safety of which would be without question.

J. C. Stevens,* M. Am. Soc. C. E. (by letter).—Mr. La Rue's plan of constructing in the Colorado River a rock-fill dam 780 ft. high and a base width of more than a mile is novel and bold. He has stated the reasons the plan should not be adopted, when he says, "this mass of loose rock would not be water-tight, which feature is the most uncertain in the plan here presented."

The writer believes, however, that his plan might well be given further investigation with a view to modifying the design and method of construction so as to remove this uncertainty.

There can be little doubt that a rock-fill dam to the height proposed will stand throughout the ages and will not involve the uncertainty of stresses that would attend the design of a solid concrete dam to this unprecedented height. The only question is whether it can be made water-tight at an expense that will give it an advantage over a solid dam.

* Cons. Hydr. Engr. (Stevens and Koon), Portland, Ore.

The writer thinks the plan of making a dam 200 ft. high by one blast from the canyon sides is somewhat fanciful. Under the conditions assumed, namely, 100 ft. to bed-rock and a water surface width of 450 ft., and, further, assuming that the canyon sides slope $\frac{1}{2}$: 1, a fill that would raise the water 200 ft. would have a projected area of 150 000 sq. ft. The finer material would undoubtedly be washed away, leaving a fill composed of rocks from the size of cocoanuts to box cars. The voids in this mass would be at least one-third, through which the water would readily pass at a velocity of 0.5 ft. per sec. Thus, a flood of from two to five times the summer flow might pass through the fill before water reached the 200-ft. level.

It would be almost impossible to reduce this leakage to safe quantities by any amount of blanketing. For instance, for a leakage of 500 sec-ft. the velocity through the mass would be about 0.003 ft. per sec., or 250 ft. per day. Water travels faster than this in natural sand and gravel beds, moraines, etc. The first blanketing of stones, gravel, and earth would tend to fill the voids somewhat in the up-stream face of the fill, but this would reduce the velocity so that no more material would be carried into the fill. The leakage thereafter would become that through the blanket itself, and, although the velocities through it could be reduced to very low values, the aggregate amount of leakage would be great on account of the enormous area exposed to the water.

It seems to the writer that a type of dam worth consideration would be a combination of solid concrete and rock-fill. By way of a suggestion: Unwater the river by means of a tunnel and a coffer-dam at the river level. Then build a solid concrete dam to a height of 50 ft. above the river surface, and let this dam form the up-stream toe of a rock-fill to the full height of 780 ft., the rock-fill to be faced with pre-cast reinforced concrete slabs anchored together to form a flexible facing to the fill and joining with the solid concrete dam at the toe. Heavy eye-bolts should extend through the slabs in both directions, by which each could be flexibly joined to its neighbors. After placing, the joints should be filled with asphaltum.

This presupposes that the rock-fill will be placed, not dumped, and that the up-stream third of the dam will be essentially a dry wall. The slopes of the fill need not be more than 1:1 on the up-stream side and 2:1 on the down-stream side. Just how much of the rock-fill should go to bed-rock, if any, could only be determined after the character of the foundations was known.

J. H. Quinton,* M. Am. Soc. C. E. (by letter).—The author is entitled to much credit for his valuable contributions to the knowledge of the physical features of the Colorado River, not only in his paper, but also in public documents.

It cannot be said, however, that he has contributed much to the knowledge of building dams in great rivers, for he proposes a plan for building a dam in the Colorado River, in which he substitutes blind force and haphazard methods founded on "beliefs", for technical skill and knowledge founded on well-known principles.

* Cons. Engr., Los Angeles, Calif.

The author's idea of blasting the loose-rock portion of a dam in place is not new, as it was tried at both the Lower Otay and Morena Dams, but was not successful. The loose-rock portion of the Lower Otay Dam was composed of a mass of quarry refuse above the steel core, and of larger rock below that core.

The author proposes to build a dam, below which the water is allowed to percolate, by blasting rock from the cliffs on both sides of the canyon. Such a dam must be of such length, up and down stream, that the underground water emerging from its lower toe will not have sufficient force to carry away the sand or silt from the river bed or from the interstices of the material composing the dam. This length depends on the kind of material composing the river bed and, for each material, has a definite ratio to the head of water above the dam. For such material as that composing the bed of the Colorado River, this ratio is between 12 and 15. The flow of water under the dam may be treated as the flow of water in a pipe filled with sand or other permeable material.

The water is never allowed to pass lengthwise through the dam, and is prevented from doing so, either by a concrete wall, or an upper slope of impermeable material. The dam itself may be either permeable or impermeable. In the former case, the surface of the lower slope must be above the hydraulic grade line at all points. In the latter, the impermeable material below the hydraulic grade line must be heavy enough to resist the upward pressure of the water, with a reasonable factor of safety.

A good example of the former is the Laguna Dam on the Colorado, in which the ratio of length to head is 13.5, and a good example of the latter is the Granite Reef Dam on the Salt River Project, built by the U. S. Reclamation Service. The Laguna Dam, however, was not built by blasting a mass of rock into the river bed, but, like all dams which have stood the test of floods, the rock was laid in place with care. Three concrete cross-walls, each 5 ft. thick, and extending across the river bed, from bed-rock on one side to bed-rock on the other, serve to hold the loose rock in place. The top layer of rock was laid to an even slope by hand and set in cement concrete. This is a diversion dam, and is only 19 ft. high, from the bed of the river to the top of the concrete wall at the crest. The dam proposed by the author, if properly designed, would have to be at least 13.5×780, or 10 530 ft. long, up and down stream, would have a volume of about 106 000 000 cu. yd., and require the blasting of at least 50 000 000 cu. yd. of rock. This material, if laid carefully on the bed of the river to form a stable dam, would probably cost $1 per cu. yd., or more.

If the rock is blasted in great masses into the canyon and the river allowed to "work its sweet will" on it during the flood season, a great deal of it would be carried down stream. Much of it would also sink into the sandy bed of the river, and in a year or two probably another 50 000 000 cu. yd. of rock would be required. As designed by the author, the dam would not withstand 650 ft. of water even if all the material composing it were laid carefully in place, for the reason that the water would fill the interstices of the material. The mass would become fluid and would not stand on a slope of 4:1 or 6:1.

It would creep gradually down stream, and as soon as the water of the reservoir commenced to pass over the top, it would be destroyed.

Some of the rock-fill storage dams which have stood the test of full reservoir and great storms are the Minidoka Dam (90 ft.), built in 1904, by the U. S. Reclamation Service, in Snake River, the Zuni Dam on the Zuni Indian Reservation, New Mexico (70 ft.), and the Morena Dam of the San Diego (Calif.), Water-Works (160 ft.).

Each of these dams has an impermeable or slightly permeable "curtain" or "cut-off", carefully connected to an impermeable and stable foundation. The loose-rock pile in a storage dam is not only a support, but also a blind drain for the slightly permeable material, and is generally of such dimensions as to withstand, alone, the pressure of the water, with a reasonable factor of safety against rupture or sliding. There is no record of a storage dam of the rock-fill type having been constructed without an impermeable cut-off connected to an impermeable foundation.

In this type of rock-fill, it is considered unsafe for the loose-rock part of the dam to rest on any but an impermeable foundation, as there may be considerable leakage due to imperfect work or material, in addition to the natural leakage which is to be expected through the material of the curtain. If this leakage should be sufficient to saturate the material under the loose-rock pile, settlement of the rock will occur, which would affect the stability of the dam, and eventually cause its destruction. It will be seen that to build a properly designed rock-fill dam in the Colorado River, it would be necessary to make an excavation large enough to support the rock pile on bed-rock, or other impermeable material, such as cemented gravel. The excavation alone would cost more than a masonry or concrete dam complete, and this plan, therefore, may be dismissed as impracticable.

Nothing has been said of the advisability of building a rock-fill dam 780 ft. high, even if it were practicable. A dam of such unprecedented dimensions, and in such a great river as the Colorado, would have to be of masonry and as solid as the rock cliffs above it, in order to be safe and to inspire confidence in the people living below it. The dam proposed by the author, and his method of building it, is nothing more than a dream, or a vain imagining. "The baseless fabric of this vision" is the proposition, first to blast in place a dam 250 ft. high, with a gradual lower slope extending nearly a mile down stream. Nothing is said about the upper slope of this dam, which may be supposed to adjust itself to the angle of repose for material of this nature. Even if this could be accomplished, the water would not rise to the 200-ft. level, because it would flow freely through the interstices of the rock which forms about 50% of the mass (instead of 15% as stated by the author), and would cease to rise when that flow equalled the flow of the river above the reservoir.

The writer feels that he has said enough to show that the dam proposed by the author is not practicable. Great engineering structures cannot be founded on "beliefs" unless there are "cold facts" to justify them, and none of the latter is found in this paper. It seems unfortunate that so many people,

even those in high places, should have expressed their belief that a loose-rock dam could be built by blasting cliffs into a canyon of the Colorado River, and it is hoped that this discussion will put an end to this foolish idea. Such ideas expressed by engineers, when not backed up by facts and knowledge, only cause ridicule among those who have had experience in building dams in great rivers, and give the general public a wrong impression which is sometimes difficult to eradicate.

ERNEST H. BALDWIN,* M. AM. SOC. C. E. (by letter.)—The magnitude of the proposition for damming the Colorado River, advanced in such an interesting manner by the author, would be sufficient in itself to attract the attention of engineers interested in this branch of engineering, but this feature, combined with many other data relative to this important but perplexing river, makes this paper extremely valuable.

The general proposition of a high dam on the Colorado River, whether at Lees Ferry or Boulder Canyon, is undoubtedly feasible and of prime necessity, but in constructing such a dam there should be no guesswork, no radical departure from the tried and tested methods that have proved successful and that require only expansion, and most careful planning, to insure their success in the Lees Ferry Dam.

The writer has had experience in the blasting of rock into rivers for the purpose of forming rock-fill dams, although on a scale small in comparison with the dam proposed by the author. The most carefully laid plans will miscarry in this method of building dams, and where millions of cubic yards of rock are to be placed without interfering with irrigation, a miscalculation may be more expensive than another type of dam.

It is stated that, at the site of the Lees Ferry Dam, the bed-rock is probably overlaid with 40 ft. of boulders and sand and 60 ft. of fine sand and silt. This means that several million cubic yards of the first rock blasted would settle down, as the upper 60 ft. of sand and silt was eroded, until it rested on the bed of boulders. As the head of water increased, there would be considerable scour, with consequent leakage.

Mr. La Rue states that the first shots will blow enough rock into the river to form a dam 250 ft. high, and extending to the toe, more than a mile down stream. With the settlement due to erosion, this means about 20 000 000 cu. yd. of rock, and the blast would require about 500 tons of explosives to bring the dam to the dimensions mentioned. This is an enormous quantity of rock, too much to have misplaced.

If such a blast was feasible, the rock would pile up 100 or 150 ft. from each shore, much of it fine material, but more of it in masses of 100 to 1 000 tons, depending on the placing of the mines and the manner of loading. It would not be well mixed, the fine and the coarse rock settling at different distances from the shore. The central 100 ft. or more of the river would form a channel that would scour to the boulder formation at least, and the large masses of rock would arch, forming immense caverns that could never be filled. The material for the central, or channel, part of the dam would

* Springfield, Mo.

have to be placed by another method, and the swift current would probably transport most of that material beyond the toe.

To build a rock-fill dam as suggested by the author, would mean an extensive plant for handling and placing the material, as not more than one-third of it could be blasted into place and that could not be made tight by feasible methods. The writer believes the plant required to complete the dam would be almost as elaborate and expensive as one required to build a concrete structure.

The author suggests that 10 000 000 cu. yd. of fine rock, sand, and silt might be required to tighten the dam. This, in itself, would require a large plant for loading, transporting, and placing. To be effective, the tightening material would have to be deposited in such a manner that the reservoir water would carry it at least to the center of dam, which would mean a long, tedious, and expensive process.

Should it be found, after the initial blast of about 20 000 000 cu. yd. of rock, that conditions were as previously suggested, then several million dollars would have been wasted and a good dam site spoiled.

In the writer's judgment, a dam 780 ft. high could be more economically constructed of concrete if the foundation was found to be suitable. The natural conditions at and adjoining the site seem to be favorable for the making of aggregate and the placing of concrete.

C. S. Jarvis,* M. Am. Soc. C. E. (by letter).—The proposed plan for a 780-ft. rock-fill dam on the Colorado River is startling in many of its details, but the character and achievements of the author are such as to command a more serious consideration than would be accorded otherwise.

The remoteness of the proposed site from industrial centers might be urged against the project; but the idea of such a type of construction should be examined on its merits, for there are many dam sites more favorably located, with the same characteristics, but generally of smaller dimensions, than the one described at Lees Ferry. The question is mainly as to the practicability of such a dam, as the author specifically states in his introductory remarks.

The transporting power of running water is fairly calculable; but formulas do not take into account the action around the larger boulders. These may appear in the bed of the stream, partly submerged in rock waste or silt, and, when the flood is at its height, erosion will occur around such an obstruction. When a sufficient space has been cleared laterally and down stream, the boulder may slide or roll a short distance, and the alternate scouring and movement will be repeated. During the receding stage of the flood, the hollows down stream from the boulders are generally filled in part, so that an observer might consider such large stones safe against movement, if he neglects the effect of erosion, as described. This relentless scouring adjacent to obstructions is the element most directly opposed to the type of high rock-fill dam under discussion.

There is no doubt that, with suitable preparation and blasting, large enough masses of rock could be dislodged to create a loose rock-fill; but, to the writer,

* Civ. and Cons. Engr., Salt Lake City, Utah.

it appears improbable that a mud bank would form up stream and "make the rock-fill water-tight to an elevation of 50 to 100 ft. above the bed of the river"; that such a bank could be raised ahead of the rising water surface of the reservoir so formed; or that the rock-fill could be made reasonably water-tight in the midst of such a river, if such a mud bank should be formed and then overtopped at a height of 100 ft. If 20 000 000 cu. yd. of rock were thrown into the river gorge, as outlined by the author, the rapids formed thereby would exert their power to remove the obstruction, eroding at the down-stream toe, and rolling or sliding the protruding boulders into the hollows thus formed, until the rock-fill had spread, settled, and displaced sufficient of the river sand and gravel to insure stability for that particular stage of flow. A higher flood stage would probably cause a resumption of the process of erosion and shifting. During the low-water stage, the entire flow of the river might run through the rock-fill, and the erosion and transportation of the finer materials would continue, mainly from the up-stream section of the dam.

The writer served as engineer on one irrigation project where time limits and flood conditions required the placing of a 20-ft., concrete, diversion dam partly on bed-rock and partly on a rock-fill in the midst of a rapids in the river channel. It was well known that leakage would result, with no danger to the structure. A rock and earth-fill dam was to adjoin the concrete section and extend to the opposite bank on and immediately up stream from the rock-fill in the river bed. The structures were built during flood stages of the stream, and maintained safely; but to stop the recurrent and persistent leaks through this rock-fill required the placing of an additional volume of finer materials about equal to the volume of the structure as originally planned; and, finally, during a low stage of the river, a cut-off wall was placed through a part of the river bed, at the up-stream limit of the dam, to prevent the recurrence of leaks and slow erosion of the finer materials.

The construction of several rock-fill dams with puddle cores on glacial drift, together with observations on these structures in service, and the lateral and terminal moraines which serve as the retaining embankments for reservoirs and lakes of great depth, have convinced the writer that a combination of rock-fill and hydraulic-fill may be built and maintained safely at such a height as the author has suggested; but a pre-requisite for making it reasonably water-tight and dependable would be the positive checking of the underflow and the control of the river during construction, to prevent the segregation and loss of the finer materials.

Those who have had to construct measuring weirs in rocky stream beds with the water at high stage need not be reminded of the difficulty in making the structure water-tight, especially at the junction with the stream bed. The volume of finer materials required to complete the dam that holds back a depth of water of only 2 or 3 ft., may be doubled or trebled, because of the persistent erosion and loss of the finer particles through the rock-fill.

The extensive studies and writings of the author on the utilization of various Western rivers, and his location of the most feasible dam sites and storage basins, are valuable contributions to professional knowledge.

Referring to Conclusion No. 9, "Should either the rock-fill or a concrete dam be overtopped by a flood, the damage might be sufficient to cause failure", it seems improbable that a concrete structure of such magnitude would be designed or constructed so as to permit failure if overtopped, whereas the rock-fill would certainly be lost under such a condition, and the resulting damage and loss in the lower valleys would be incalculable.

In view of these considerations and the uncertainties that would envelop the construction, the cost, and the behavior of the rock-fill dam as proposed, it seems a safe conclusion that any dam of unprecedented height at the proposed or similar sites will be of masonry, founded on bed-rock, or of rock-fill with adequate core-wall.

SAMUEL FORTIER,* M. AM. SOC. C. E. (by letter).—The people of the West are much interested in feasible plans to utilize the waters of the Colorado River. Even a partial utilization of this river might affect for good or ill the citizens of seven Western States. Perhaps the most valuable part of the author's paper on this subject is in outlining the wonderful potentialities of this river, when properly controlled, in furnishing water for irrigation, producing energy, and protecting fertile lands from floods. He might have added a fourth, namely, the settlement and retention of the river sediment within the natural channel of the river.

For several years, the U. S. Bureau of Public Roads, under the direction of the late C. E. Tait, M. Am. Soc. C. E., has been carrying on an investigation of the silt problem of the Colorado River, in co-operation with the State of California and the irrigation interests of the Imperial Valley of California. As a part of this investigation, daily samples were taken at Hanlon Station, the river intake of the Imperial Canal System, from July 1st, 1917, to June 30th, 1918, to determine the percentages of silt entering the System from the river. These percentages were ascertained by filtering out and weighing the solids contained in the muddy water of each sample.

In Table 4 is given the maximum, minimum, and average daily percentages by weight for each month of the period covered.

Owing to the uncertainty in ascertaining when silt has reached its least volume in the process of consolidation, it is more difficult to express with the same degree of accuracy the percentage of silt by volume. From October, 1907 to September, 1908, samples of equal volume were taken daily from the turbulent water below the gates at a number of stations of the Imperial Canal System and poured into one container for each calendar month. The composite samples thus obtained were then shaken and settled for 30 days in glass tubes, 1 m. high, after which the percentage of silt was noted. The results obtained on the Main Canal at Hanlon Head-gate was a maximum of 3.8% in August, a minimum of 0.9% in July, and a mean for the year of 1.6 per cent. If due allowance is made for differences in the quantity of silt carried in 1907-08 and 1917-18, a percentage of 1.6 by volume would compare in a general way with a percentage of 0.242 by weight.

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In 1919, there was delivered to the Imperial Irrigation District 1 584 312 acre-ft. of water diverted from the Colorado River, and this quantity of water was applied to 413 440 acres of land. These figures do not include the quantity of water delivered to lands in the Republic of Mexico, served by the same system. In that year, the cost to the Irrigation District and to the several subsidiary companies known as mutual water companies for removing silt from the canals, amounted to approximately $1 000 000. This cost did not include that incurred by the farmers in removing silt from farm laterals, damages to crops as a result of silt, or modifications necessary on account of yearly accumulations of silt on cropped fields. This extra cost incurred by the farmers may be conservatively estimated at $2.00 per acre per annum, thus bringing the total damages caused by silt in one year to $4.42 per acre irrigated.

TABLE 4.

Date.	Maximum percentage by weight.	Minimum percentage by weight.	Average percentage by weight.
July, 1917	0.750	0.063	0.232
August, 1917	1.033	0.174	0.574
September, 1917	0.516	0.074	0.145
October, 1917	0.990	0.129	0.371
November, 1917	0.249	0.060	0.111
December, 1917	0.159	0.064	0.080
January, 1918	0.217	0.050	0.088
February, 1918	0.219	0.036	0.083
March, 1918	1.309	0.121	0.609
April, 1918	0.540	0.103	0.286
May, 1918	0.322	0.140	0.266
June, 1918	0.269	0.019	0.061
Mean			0.242

If it is true, as reported, that 1 220 000 acres are irrigable from the lower part of the Colorado River, the damage caused by silt in one form or another, unless much better controlled than at present, would impose yearly a heavy financial burden on the part of the water users.

It is evident, therefore, from what has been stated, that the waters of the Colorado River should be stored not only to provide water for irrigation, develop power, and prevent floods, but also to retain the river silt. The cost of such storage on a large scale by concrete dams is not likely to exceed $2.25 per acre-ft. The quantity of water required to irrigate such crops as are grown on the Lower Colorado Basin is usually less than 3 acre-ft. per acre, measured at farmers' deliveries. Accordingly, the first cost of storing enough water to irrigate 1 acre of land in this Basin, exclusive of transmission losses, would be about $6.75, as compared with $4.42, the present annual cost of removing silt from channels and compensating land-owners for the injurious effects of depositions of silt on irrigated land.

It will be noted by a reference to Table 4, that the average percentage of silt by weight is 0.242. The waters of the Colorado River at Yuma, Ariz., contain slightly more than double this quantity, or 0.5 per cent. The differ-

ence is due to skimming and other processes by which the water admitted to the main canal of the Imperial System is rendered freer of solid matter.

As to the author's tentative plan to store water at Lees Ferry, the writer regrets that he cannot approve of the site selected or the plan proposed. The site is too far removed from available irrigable lands, settled communities, or transportation facilities. Owing to the long distances between the dam site and the markets for power, only a small percentage of the hydro-electric energy which might be developed could be utilized. Besides, as the writer has tried to point out, the retention of silt is an important factor in storing water, and by locating the reservoir at Lees Ferry the tributaries which carry the largest quantity of silt would enter the main river below the reservoir.

As regards the author's tentative design for a rock-fill dam in the channel of the Colorado River at Lees Ferry, the feasibility of blasting enormous quantities of rock at a low unit cost into the channel from the adjacent side-walls in the manner proposed will not be questioned. To render water-tight such a quantity of blasted rock so placed, in the writer's opinion, would be impractical in any such manner as that proposed. The author states that the loose rock in the dam would be about 15% greater than the space occupied by the same material in its natural compact state. This is evidently an error, as authorities are agreed that when 1 cu. yd. of solid stone is broken into pieces, it will occupy when loose about 1.9 cu. yd. In other words, the voids in broken rock generally run from 40 to 50 per cent. In blasting rock in the manner proposed, a fairly large percentage would be thrown into the river gorge in large, irregularly shaped masses, some of which might contain 1 500 cu. yd. When piled up on the site of the dam, these large masses would contain correspondingly large open spaces. There is a further possibility that these large open spaces would be connected, thus permitting large quantities of water to pass through, especially if the water was under high pressure. The author proposes to render such a mass of loose rock water-tight, first by sluicing in broken rock varying in size from 1 in. to 6 in., followed by the sluicing of sand. The bulk of such rock could not well enter the larger cavities, except through the transporting power of the water. So little could be done to control the action of the water in moving the smaller rock particles from cavity to cavity that the element of chance assumes large proportions. In fact, the whole procedure would be a game of chance. Perhaps the rock particles might find a lodging place, and then, again, perhaps they might be borne through the structure by the force of high heads of water. In any event, the greatest efficiency that could be expected would be that a relatively small part of the voids in the rock-fill could be filled in this way. The author also proposes to sluice sand into the open spaces when the latter are filled with broken rock. The use of sand, however, is based on the belief that the cavities would first be filled with the smaller broken rock. Otherwise, the sand would be transported through the rock-fill.

A Combination Concrete and Rock-Fill Gravity Dam.—Although the writer believes that it would be unsafe as well as unwise to build a loose rock-fill dam in the manner outlined, and that if a dam is built in Boulder Canyon, or elsewhere on the Colorado River, it will be of concrete through-

out, he is of the opinion that a certain percentage of loose rock, having the open spaces well filled with finer material, might well form part of a concrete gravity dam of the dimensions under consideration. The foundation of a combination structure of this type, up to the natural high-water elevation, would be laid in concrete in the manner customary in building gravity dams, but the superstructure would consist of massive concrete walls enclosing compartments filled with broken rock, gravel, sand, and rock dust. By exercising care in grading and in placing the rock materials, the specific gravity of the loose rock in place could be made to approach 80% of that of the concrete, whereas the cost per unit of volume of rock-fill in place, as compared with concrete in place, would be about as 1:3.

This suggestion is merely a new adaptation of an old idea. From the days of the Pilgrim Fathers, the pioneers of every State in the Union have made use of wooden crib-dams filled with rock. Unless the native logs or sawn timbers, of which the cribs are built, are continuously submerged, early decay is inevitable. In these days of higher cost of timber, as compared with that of concrete, this old type of pioneer dam might well receive renewed attention, providing concrete, either plain or reinforced, can be used in lieu of timber. For example, some advantages in the way of lessened cost might be derived in using this combination structure for relatively low diversion dams by building on a secure foundation a skeleton structure of reinforced concrete, enclosing compartments to be filled with loose rock. At the other extreme, as regards the height and other dimensions of dams, is to be found such a structure as is contemplated for one of the canyons of the Colorado River. Nothing approaching any such structure in size has ever been built, and the internal stresses produced by the setting of the concrete and changes in temperature would be more or less conjectural; whereas in building what might be termed a skeleton dam of concrete, but of ample strength, the thickness of the several longitudinal ribs would be well within established practice. The general form of such a type of dam would not differ materially from the common gravity type, except that the dimensions would have to be increased to take care of the lesser specific gravity of the loose-rock portion as compared with concrete. The up-stream face would also consist of a series of sharp arches abutting against the vertical longitudinal ribs and against the side-walls of the canyon. These ribs, in turn, would be tied into arched cross-ribs, thus enclosing compartments of the general form of the frustum of an inverted pyramid. Such a form would tend to wedge in and compress the loose rock placed therein. A drain to bed-rock could be installed in the base of each compartment, and provision made to drain each compartment. There would be no horizontal partitions in any compartment, but the top of each might be sealed with reinforced concrete if found to be desirable.

The main advantage in using loose rock in conjunction with concrete, in a dam of the kind proposed, lies in the lessened cost. If such a structure could be designed to consist of 40% loose rock, the saving in first cost would be considerable. Other minor advantages might lie in greater elasticity and better facilities for expansion and contraction joints and draining the

foundation. The reduced shearing strength along horizontal planes and the difficulty in designing safe multiple arches on the up-stream face, however, might prove disadvantageous.

L. F. HARZA,* M. AM. SOC. C. E. (by letter).—The author proposes to build a dam about twice as high as any yet constructed, and of a type which as yet has been applied to a height only one-half as great as other types. The method proposed has been applied only to very small and unimportant or temporary dams of that type, and contemplates the formation of a storage reservoir that would be as large, if not larger, than any in existence, with valuable lands and human habitations in the lower reaches of the river. Whether such a project should be undertaken with the present knowledge of such structures hardly deserves serious consideration. Man has never been able to advance safely in the science of engineering by such stupendous strides, and this work, if it is undertaken, calls for the maximum degree of safety which engineering knowledge can provide.

As a hypothetical problem, however, serving by its very size to intensify the study and discussion of this type of structure, it should prove valuable in bringing out all the experiences, theories, and opinions bearing on this type of dam, for subsequent application to smaller structures.

In 1914, the writer proposed such a plan, on a smaller scale, as one of several alternative plans, although not the favored one, for closing the main channel of the Columbia River near The Dalles, Ore., after having completed a by-pass channel and gates of sufficient size to pass and control the entire flood flow.†

In this instance, the available rock-fill material was partly to be piled for this purpose behind large rubble retaining walls and was assumed to cleave on a plane inclined on a slope of 1 on 1, after the blasting of the retaining walls, thus furnishing sufficient material (about 900 000 cu. yd.) above this slope to divert the river completely.

The writer knows of one instance where a blast on a smaller scale was used to close a narrow river gorge under a head of about 100 ft. Subsequent attempts, extending over about 7 years, to seal this rock-fill with fine material have not proved successful, unless it has been accomplished recently. Local settlements of the fill occur periodically, which settle and crack the blanket of fine material, starting channels of flow which quickly enlarge until a hopper-like cavity forms in the blanket, through which water escapes freely into the coarse rock.

The writer believes that the principles which should determine the design of the blanket to tighten such a rock-fill dam are:

(*a*).—The material must be graded from coarse to fine by such gradual transition and by such definite and controllable means, that each size of material will be continuous over the whole surface and will be too coarse to be carried away through the pores of the next larger size.

* Cons. Hydro-Elec. and Hydr. Engr., Chicago, Ill.

† "The Columbia River Power Project," Technical Publishing Co., San Francisco, Calif.

(*b*).—That each layer must be of sufficient thickness to preclude the possibility of local settlements of the supporting rock that would cause fault slips in this blanket sufficient to bring the fine material in contact with material coarse enough to permit the fine material to be carried away by percolating water, and thus destroy the blanket locally.

(*c*).—Or, preferably, that the earth blanket, in addition to Requirement (*a*), should also be of the full thickness of an earth dam if standing alone and unsupported by the rock-fill, in order that the velocity of the seepage may be comparable to that in earth dams.

Obviously, Requirements (*a*) and (*b*) are difficult or impracticable of accomplishment when material is deposited under water on an irregular surface, as would be the case if the rock-fill was formed by a large blast from the adjacent walls. It might be argued that the natural process of percolation would eventually segregate the materials as demanded by Requirement (*a*). This, however, would require many years of repairing blow-outs, settlements, and cavities formed by the carrying out of material, before equilibrium would be established, if indeed it ever would be.

Requirement (*c*), if the up-stream slope of the rock-fill was 2:1, might result in a slope of about 7:1 or 8:1 for the final up-stream surface.

If an earth dam at full section was to be built up stream from the rock-fill, its foundation would require the same treatment, prior to the blast, that would be given to an earth dam constructed by the usual means. This would require under-flow cut-offs to bed-rock, the grouting of all fissures in the bottom and side-walls to be covered by the fill, and a liberal number of concrete cut-off walls in the bottom and sides of the gorge to prevent seepage along the contact between the earth and the rock, which otherwise might cause channels that eventually would threaten the safety of the structure.

The writer would like to cite certain observations bearing on the principles of the proposed type of dam. In British Columbia, there is a narrow canyon, through which a river flows with a flood of about 50 000 sec-ft. and with a fall of about 50 ft. per mile. Just above a narrow section, flanked with steep mountains about 8 000 ft. high, the river becomes suddenly sluggish and meanders through an alluvial plain that is about ½ mile wide and 30 miles long and as flat and marshy as any point along the Mississippi. The mountains rise abruptly on each side of this marshy plain, forming a section of valley like a "V," with a straight horizontal line half way up. This condition strongly suggests a natural dam formed by faulting or by a landslide at the transition from the marshy valley to the roaring canyon. Borings have proved it to be of landslide origin. Here is a natural rock-fill dam acting as its own spillway, which has been silted successfully by natural process. The down-stream slope, however, is about 100 : 1, and the silt blanket has filled the reservoir.

The writer studied and reported* on another such instance in 1915, that of the Cascade Rapids in the Columbia River. He refers to this report because it conveys a lesson in conflict with opinions expressed in some of the

* *Bulletin No. 5*, State Engr. of Oregon.

discussions. The statement has been made that it would be difficult to reach the center of the fill with rock from the two walls. Undoubtedly, this would be the case if the rock was blasted to its resting place. If, however, it is merely dislodged along a cleavage plane so steep that the whole mass would tend to slide, the writer is not sure but that the center of the dam would be the highest.

The rock slide of the Cascade Rapids from an elevation of 3 420 ft. appears to have buried a river 1 mile away and diverted it to a new channel about 3 miles away, of which the detritus still forms the northern bank. Perhaps this is the result of the inertia of the slide and of a subsequent movement on a slippery bed. The writer also remembers having observed small avalanches with the outer nose of the detritus piled higher than that part closer to the mountain, or, at least, it so appeared without an instrument. So many plans for dams like that of the author, but on a smaller scale, have been proposed that it seems to be the natural impulse of almost every engineer, when he sees a narrow, deep canyon, to want to build a dam by the rock-slide method.

I. GUTMANN,* ASSOC. M. AM. SOC. C. E. (by letter).—It has been suggested by Mr. Muckleston that there is an analogy between the rock-fill dam, proposed by Mr. La Rue, and a natural landslip, which, in 1893, dammed the Birahi-Ganga, in India, and formed Gonha Lake. It may seem strange to compare the dam, which is proposed to be built by blasting rock from solid canyon walls, to a pile of rock formed by a natural landslip. The analogy, however, appears most appropriate and highly impressive after one learns, from the references pointed out by Mr. Muckleston, the following details of paramount importance:

1.—That the Birahi-Ganga landslip consisted of enormous masses of limestone and Dolomite rocks, mixed with detritus and covered with a "thick layer of white impalpable powder."

2.—That the rocks sheared off from a steep hill and dropped from a height of 4 000 ft. above the bed of the stream.

3.—That the 900-ft. dam thus formed stood for about a year until it was overtopped.†

There seems to be sufficient mechanical similarity between slides of this type and the new method of dam construction suggested by Mr. La Rue, to justify further reports and studies of dams formed by natural landslips.

The writer can cite a similar occurrence, on a much smaller scale, which took place on the Jordan River, in Palestine, on December 8th, 1267. According to an Arab chronicle,‡ a hill on the west bank, near Damiah, fell into the river and kept it completely dammed for sixteen hours. The Jordan was in full flood at the time. From a direct knowledge of the river, the writer would think that the river may have carried from 2 000 to 3 000 sec-ft. The natural dam must have been composed of alluvial and diluvial earths.

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* Elec. Bond & Share Co., New York City.

† *Journal*, Soc. of Arts (Lond.), March 27th, 1896.

‡ "From the Garden of Eden to the Crossing of the Jordan", by Sir William Willcocks, Lond., 1920.

E. C. La Rue,* M. Am. Soc. C. E. (by letter.)—The writer's paper was prepared before detailed surveys had been made of the Lees Ferry project. It was necessary, therefore, to estimate the capacity of the reservoir, the depth to bed-rock, distance between canyon walls at the dam site, length of spillway tunnel, etc. The results of detailed surveys and diamond drill borings are now available, and it is believed that definite information relating to the Lees Ferry project will be of interest. Table 5 gives the figures disclosed by the survey in comparison with those estimated by the writer.

TABLE 5.—Data Relating to the Lees Ferry Project.
(Preliminary Estimates and Data Disclosed by Surveys.)

	Preliminary estimate.	Data disclosed by surveys.
Width of canyon at water surface, in feet	450	430
Width of canyon at Elevation 720 above water surface at dam site, in feet	1 000	1 600
Width of canyon at top, in feet	1 800	1 900
Length of spillway at Elevation 200 above water surface, in feet	3 000	2 900
Storage capacity of reservoir at Elevation 700 above low-water level, in acre-feet	50 000 000	54 000 000
Capacity of reservoir between Elevations 600 and 700 above low water, in acre-feet	20 000 000	21 000 000
Depth of bed-rock in river channel at dam site, in feet	100	Less than 100

The greatest error was made in estimating the width of the canyon at Elevation 720 above the water surface at the dam site, the writer's estimate being 1 000 ft., whereas the correct distance is 1 600 ft. The writer's estimate of the width of the canyon at the top was nearly correct, which indicates that the upper face of the canyon walls is more nearly vertical than was assumed. The volume of the rock-fill dam would be greater, but the conditions for blasting the material from the upper part of the canyon walls are more favorable owing to the fact that the upper face of the wall is nearly vertical.

Those who have studied the paper have gained the impression that the writer was advocating, a 700-ft. dam at Lees Ferry to store 50 000 000 acre-ft. of water as a project feasible of construction at this time. The primary purpose of the paper was to bring out engineering opinion on the practicability of constructing a dam such as that described. The Lees Ferry site was selected in order to illustrate the methods of construction suggested, because it was well known to the writer and seemed to offer unusually favorable conditions for the building of such a dam. The writer was not and is not advocating the Lees Ferry project or any other specific project on the Colorado River as the proper one to be developed at this time. During the past seven years, he has frequently stated that before any money is spent on the construction of a high dam on the Lower Colorado, an investigation of all possibilities should be made, and a report should be prepared by competent engineers, outlining a comprehensive plan for the development of the Colorado River Basin from Wyoming to the Gulf of California. He has also explained that the first large project to be undertaken on the lower river, should be a unit of a carefully prepared compre-

* Hydr. Engr., U. S. Geological Survey, Pasadena, Calif.

hensive plan for the development of the whole basin. In the writer's opinion, sufficient information is not available to form the basis for such a comprehensive report, and he will state further that no report based on detailed engineering data has ever been prepared to show the most economical plan for the development of the Colorado River below the mouth of Green River.

When complete data are available, an intelligent conclusion can be reached as to whether it will prove most economical to develop the first large flood-control reservoir at Lees Ferry or elsewhere. It will then be possible to evaluate properly the advantages of creating a storage reservoir where it will control this great river for comprehensive power development through a fall of more than 2 000 ft. and simplify the spillway plans and construction problems of eight or ten dams, and where it will afford adequate regulation for irrigation and the prevention of flood damage, and weigh them against the advantages of a reservoir that may control 10% more water at the foot of the power section of the river and only half as far from the principal area of irrigable and flood-threatened lands. The real value of creating a storage reservoir at the head of the fall at Elevation 3100, which would make possible the development of between 3 000 000 and 4 000 000 h. p. between Lees Ferry and Black Canyon, and result in a saving of millions of dollars in the construction cost of all dams built in the river below, would be disclosed.

Mr. Coutlee has called attention to a number of rock-fill dams in Canada. It is interesting to note that a rock slide, 2 miles long and 2 miles wide, on Old Man River did not act as a water-tight barrier, it being stated that without gathering much head the small river was able to seep through the dam. Mr. Coutlee appears, however, to hold the opinion that satisfactory results can be obtained by the construction of dams of the rock-fill type.

Mr. Muckleston calls attention to a natural rock slide that occurred in 1893 on the Birahi-Ganga, a small stream in India. Although this natural dam was 900 ft. high, 11 000 ft. wide at the base, and 2 000 ft. wide at the top, Mr. Muckleston states: "Nevertheless, the dam went out as soon as it was topped." He failed to state that the natural dam had no spillway and that due to this fact it was known that the dam would fail.

The writer believes that the paper describing the failure of the Gohna Lake Dam contains information of great value to engineers interested in determining the feasibility of constructing high rock-fill dams. The following quotation was taken from a paper entitled "The Great Landslip at Gohna, in Garhwal, and the Measures Adopted to Prevent Serious Loss of Life",* by J. H. Glass, Chief Engineer and Secretary of the Department of Public Works of Bengal:

"On the 19th of November, 1893, the first intimation of the Gohna landslip reached the Government. * * * The district authorities had heard from a native source that early in the previous September an enormous mass of material from the Maithana hill had subsided into the Bireh Ganga river, completely blocking it, and that a large lake was under course of formation. For some time afterwards material continued to fall, huge blocks thundering down

* *Journal*, Soc. of Arts (Lond.), March, 1896, p. 431.

the hill, and crashing into the mass below, but there was no further fall of any size till about the end of October, when a great slice of the hill moved forward into the valley, making an effective barrier of enormous proportions, and completely obstructing the flow of the river. The falling mass was accompanied by a deafening noise, and the neighborhood for miles around was filled by clouds of dust, which whitened the ground and tree branches like snow. Gohna is situated away from any road route, and is most difficult of access, so that it was not possible to obtain readily any accurate account of what had really happened. * * *

"The result of Colonel Pulford's inspection was made known to Government in a semi-official letter, dated the 21st December. He reported that the mass of earth and rock composing the landslip was about 900 ft. in height, 2 000 ft. across at top, 11 000 ft. at the base along the valley, and 3 000 ft. at top and 600 ft. at bottom across the valley. These figures give an idea of the magnitude of the slip. It was largely composed of enormous masses of rock, some of which, lying on the surface, were calculated to be many tons in weight, with an admixture of detritus and a thick layer of white impalpable powder over all. The slip had taken place from a precipitous hill bordering the river on the right bank. The hill, which is called Maithana, has an elevation at the summit of about 11 000 ft. above sea-level, and the part from which the fall occurred is about 4 000 ft. above the bed of the stream. The force of the fall from so great a height as 4 000 ft. carried the rocks and *débris* right across the valley of the river, and for some distance up the steeply scarped hill on the left bank; then, its energy expended, the mass slipped back into the valley, forming a dam, with a big slope up against the hill on the left bank. The further great slip in October filled up the dam on the right bank against the hill on that side, in such a manner that the top of the dam had a large depression some 150 ft. below two sloping mounds of rock and *débris*.

"The bed of the Bireh Ganga river slopes at about 250 ft. per mile at Gohna, and is the bottom of a narrow gorge, with steep, and sometimes precipitous sides. The gentler slopes of the hills are grass-covered at the base, and, higher up, are clothed with evergreen, oak, fir, and rhododendrons. Immediately below the slip, the course of the river turns nearly at right angles, and passes through a narrow gorge with precipitous sides of rock. In the more open parts of the valley, the slopes are terraced, with great labour, for cultivation; but the configuration of the ground and the nature of the soil are such as to prohibit any large amount of cultivation. The river basin at Gohna, draining into the lake formed by the landslip, has an area of about 90 miles, and is bounded on the north and east by a snow-clad ridge of the Himalayas, rising to an altitude of 21 200 ft. above sea-level. A large portion, therefore, of the water of the river is derived from the melting of the snows, and this supply is, of course, at its greatest during the hot season. * * *

"From the information then before him, Colonel Pulford fixed the date on which the water would rise to the top of the dam as the middle of May. Later on, however, when data in regard to the area enclosed by the landslip, and in respect to the rainfall and snowfall during the year over the catchment area of the river had been collected by the Public Works Officer who was sent to Gohna for that purpose, Colonel Pulford reported that the lake would probably overflow about the middle of August; and the 15th of August was accordingly adopted as the date by which all arrangements for dealing with the flood should be in working order. * * *

"Before the end of July all the arrangements which had been ordered for the protection of life and property were completed. A good deal of percolation was noticeable at the toe of the dam. * * * The area of the lake when the water was level with the top of the dam was calculated from the contoured map prepared by Lieutenant Crookshank, R. E., to be about 37 000 000 sq. ft.

in round numbers, or say 1¼ sq. miles, and the total contents were roughly estimated at 16 650 000 000 cu. ft. * * *

"The first symptom of the approaching collapse occurred on the 10th August. For some time past the water had been rising very steadily, owing to the rains which had commenced early in June and the melting of the snows, and on the 4th August it had reached within 82 ft. of the lowest part of the dam. On the 9th August there was a very heavy rain which caused a serious slip in the down-stream face of the dam, leaving an almost perpendicular section some 400 ft. high. The section thus exposed disclosed a layer of stones at the top, below which only pulverized rock was visible. Percolation, which was now very heavy, was running in a considerable stream over the boulders of the former river-bed laid bare by the slip referred to. This slip caused the death of a man called the Gohna Fakir, his wife and three children. * * * On the 11th August the first warning was telegraphed to all concerned, intimating that the lake would escape within fifteen days' time; and this was followed by a notice that the lake was 50 ft. from the top of the dam. In all 87 warning messages were despatched from Gohna. Percolation was still steadily increasing. On the 10th July it was calculated to be 50 cu. ft. per sec., on the 17th July, 150 cu. ft., and on the 12th August, 350 cu. ft. per sec. On the 22nd August, Lieut. Crookshank issued the warning message that the lake was expected to escape within 48 hours. By cutting a very narrow channel 2 ft. 6 in. deep in the crest of the dam he expected to bring in the flood early on the morning of the 24th August, but owing to increased percolation and smaller rainfall than on previous days, the lake rose so slowly that it did not reach the bottom of the cutting until the afternoon, and as it was considered important that the flood should occur during day-light, the channel was filled up so that the over-topping might take place on the morning of the 25th idem. The lake water commenced to trickle over the dam through the cutting at 6.35 A. M. on the 25th August. At first the destructive action of the water was very slow, and the volume escaping from the lake over the dam did not exceed 200 cu. ft. per sec. up to 3.30 P. M., at which time a thick mist descended and heavy rain fell preventing Lieut. Crookshank from taking further observations. But at 2 P. M., the percolation which had now increased to an alarming extent, caused a sudden cutting back of the dam on the down-stream side to within 1 000 ft. of the overflow point. At 2.30 P. M., a message had been despatched down the line to the effect that the dam was cutting back, and that the lake was expected to commence falling during the night. The thick mist and heavy rain still continued to obscure the slip, and it was quite impossible to observe the action of the water on the dam. At 11.30 P. M., a loud crash was heard, and in spite of the mist and rain the air was filled with a fine dust which rose from the dam. On observing this, Lieut. Crookshank went with the civil functionary, Palwari Mali Dasauli, down to the dam, and made a final inspection of its condition. It was then midnight. As far as he could ascertain the dam was completely breached, and the lake water was escaping through a rapidly increasing channel. The effect on the dam had already been very marked, and the upper surface where Lieut. Crookshank and the Palwari were standing suddenly sank some feet. At twenty minutes after midnight another warning message was sent down the line to the effect that a very great rush of water was being discharged from the lake. It was not possible to gauge the discharge owing to the thick mist which prevailed and the torrents of rain which were falling, nor could it be ascertained at what rate the lake was emptying. At 5 A. M., although the valley was full of mist and the light very scanty, it became evident that a great fall had taken place in the level of the lake, and the irrigation officers and civil authorities at Hardwar were warned to expect a great flood. The sides of the lake, after it had emptied below high-water mark, were waste and desolate. They were cracked and had slipped in many places, and were devoid of trees and vegetation, and an offensive smell

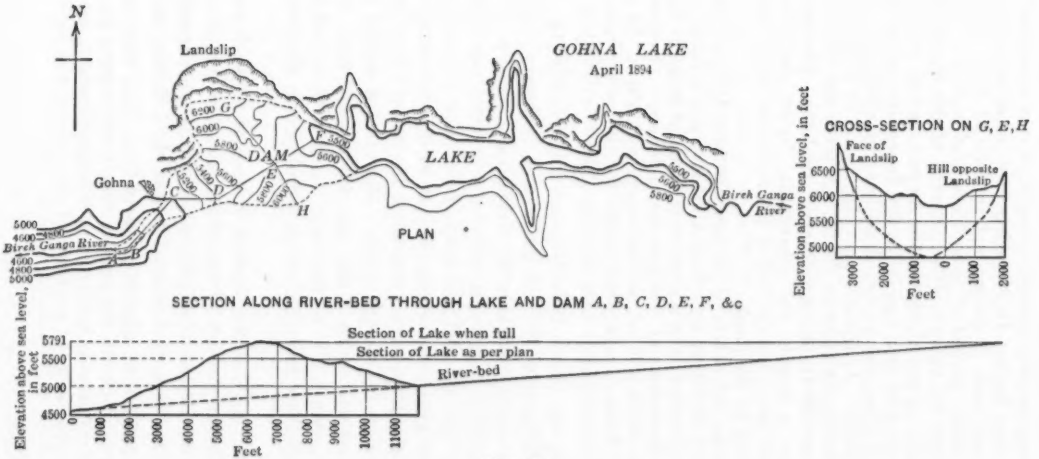

N
Landslip
GOHNA LAKE
April 1894
DAM
LAKE
Gohna
Birch Ganga River
PLAN
CROSS-SECTION ON G, E, H
Face of Landslip
Hill opposite Landslip
Elevation above sea level, in feet
Feet
SECTION ALONG RIVER-BED THROUGH LAKE AND DAM A, B, C, D, E, F, &c
Section of Lake when full
Section of Lake as per plan
River-bed
Elevation above sea level, in feet
Feet

FIG. 14.

rose from the ground which had been so long submerged. Below the dam, the valley of the Bireh Ganga was filled up with the huge blocks of rock, washed away from the landslip by the torrent, and it was found that at the gorge below Gohna, the level of the bed of the stream had been raised about 234 ft. The discharge from the lake was still going on, but was confined to the natural drainage of the river basin. In emptying itself of so large a part of its contents, the lake had made a strong and substantial weir, composed of huge blocks of rock, tightly packed with boulders, and having a long gentle slope stretching far down the valley. The first rush of the impounded water commenced about 11.30 on the night of 25th August, and by 4 o'clock on the following morning the lake level had fallen 390 ft., equivalent, in round numbers, to a discharge of about ten thousand millions of cubic feet in the space of four and a half hours. At Chamoli, a village some 13 miles from Gohna, the flood first began to rise at 11.55 on the 25th August, and the maximum height attained was about 160 ft. above ordinary flood level. The abutments of the permanent bridge, a temple, the Dharmsala, Bazar, and Dispensary, were completely washed away, and the bed of the river was raised about 50 ft. with the *débris* deposited by the flood. * * *

"The large rocks and *débris* of the landslip have been so consolidated and packed by the flood that they now form a massive and solid dam, which is not likely ever to be disturbed, and a permanent lake has probably been established at Gohna. The dimensions of this lake are as follows: Greatest length about 3 900 yd., average width, 400 yd., and greatest depth 300 ft. It may be that in time Gohna will become a pleasant resort for the weary traveller from the plains, and that boats will sail upon the lake. But it will be long before then!"

Data regarding the Gohna Lake Dam are given in Fig. 14. Fig. 15 shows the variation in the seepage through the dam as the head of water on the dam increased. In the paper describing the Gohna Lake landslip, a landslip on the lower face of the dam is mentioned, which it was stated was caused by a heavy rain. When this slip occurred, the head of water on the dam was about 840 ft. It is the writer's opinion that the slip was what is commonly called a "blow out." No doubt the down-stream section of the natural dam was too tight to permit free drainage of seepage water. However, it should be noted that even after the blow-out occurred the dam did not fail until it was actually overtopped. This dam would be standing to-day if there had been adequate spillway, and conditions favorable for free drainage in the down-stream part of the dam.

Although regretting that Mr. Warner, instead of presenting serious discussion based on his many years of experience, has limited himself largely to facetious criticism, the writer finds solace in the apparently unqualified approval given to the suggested rock-fill dam in the following sentence:

"The present and future value of property below Lees Ferry makes imperative the adoption of a dam of the gravity type [and it must be admitted that a rock-fill dam pre-eminently conforms to this specification], because it is the only type which will have the approval of the farmer, the bond-buyer's engineer, and the Federal and State regulatory bodies."

Mr. Noetzli holds the opinion that if the loose rock-fill were overtopped during construction by a flood of 100 000 sec-ft., only "a few of the largest blocks of the dam would stay within the area of the foundation." To support his opinion, he calls attention to the fact that the Lower Otay rock-fill dam was swept away when overtopped by a flood in 1916. The Lower Otay Dam

had a steel core supported by loose rock laid to a slope of 1½ : 1 on both the up-stream and the down-stream face. If this rock-fill had been constructed with its down-stream face on a slope of 14 : 1, it seems probable that, after the failure, some of the material would have been found within the original area of the base of the dam. At Elevation 250, the proposed rock-fill dam at Lees Ferry would have a down-stream slope of 14 : 1. However, precaution should be taken to prevent a flood from passing over the rock-fill either during construction or after the completion of the dam.

Mr. Noetzli apparently is not opposed to the rock-fill type of dam if means other than those suggested by the writer are used to make the dam water-tight. As Chairman of the Arizona Engineering Commission, the writer visited the site of the Parker Dam on the Colorado River and believes that a safe dam

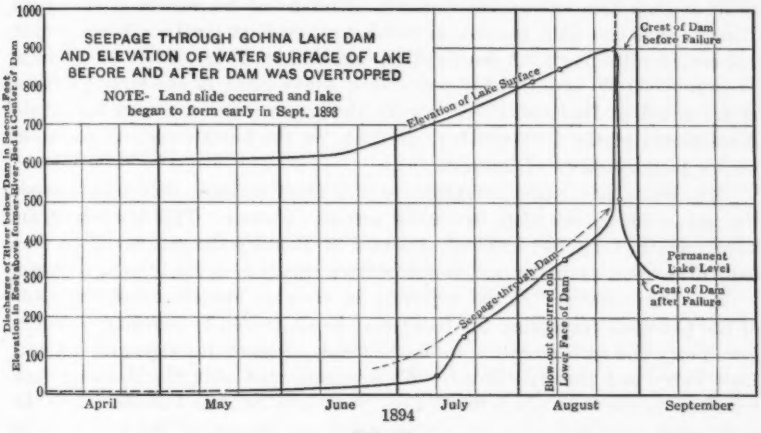


FIG. 15.

of the height proposed by Mr. Noetzli can be built at that point at reasonable cost. The combination concrete rock-fill dam proposed by Mr. Noetzli for the Parker site may be the proper solution of the problem.

The writer wishes to express his appreciation of the discussion prepared by Mr. Bryan giving the geological setting of the Lees Ferry dam site. The data submitted by Mr. Bryan will prove of value to the engineers who are finally called on to determine the most feasible type of dam to build at that site.

In proposing to build the dam to a height of 250 ft. with the first blast, the writer was of the opinion that the rock-fill would displace the fine sand and silt and settle to the layer of sand and boulders. It might be safer, as a first step, to blast a smaller quantity of material into the river, allowing the flood water to wash out the fine material which would cause the rock-fill to settle to a firm foundation. This change of plan would meet one objection mentioned by Mr. Davis, who suggests a concrete pavement cemented to the abutment walls to prevent leakage between the rock-fill and the canyon walls. Owing to vertical jointing, the canyon walls at the dam site are not smooth. If leakage

should occur between the rock-fill and the abutment walls, the water must travel a comparatively long distance, perhaps 1½ miles near the base of the dam. It does not seem reasonable to believe that such leakage would take place if the voids in the loose rock can be filled with a fine material.

Unfortunately, Mr. Davis has assumed that the writer holds the opinion that a dam 780 ft. high should be built as a first step in the plan to control the flood flow. A dam to raise the water 700 ft. was chosen, because it is what may be considered the maximum development feasible at the Lees Ferry site. A higher dam would affect the dam proposed below the mouth of Green River. The writer's opinion on this matter is, that no engineer is justified in presenting a project for flood control as being the most feasible, until he has first collected the necessary engineering data to show the possibilities for development at each dam site on the Colorado River below the mouth of the Green River. Sufficient data are now available to show that the Lees Ferry project has considerable merit. A flood-control dam at Lees Ferry, about 400 ft. high, to store 8 000 000 acre-ft. of water, with a power plant in the Grand Canyon at the mouth of Diamond Creek, may be the proper plan for a start toward the development of the 4 000 000 h. p. available on the lower river. A report on such a plan would be of interest.

Mr. Davis has called attention to the great damage that would result through suddenly releasing 50 000 000 acre-ft. of water. This is obvious, the same quantity of water suddenly released at Boulder Canyon would be even more disastrous, as this project is nearer the property most likely to be damaged.

Mr. Davis states that the writer is in error in claiming that the floods of the Colorado (excepting the Gila) may be controlled by building a dam at Lees Ferry and calls attention to the 50 000 sq. miles of drainage area between Lees Ferry and the Gila River. We are concerned with the drainage area tributary to the Colorado between Lees Ferry and Boulder Canyon. It is the flood water contributed by this drainage and not the size of the basin that is important. Mr. Davis has presented no data to support his contention that dangerous floods originate in the basin between Lees Ferry and Boulder Canyon. The Little Colorado is by far the most important tributary to the Colorado in this section. Intermittent records are available, showing the flow of this stream since 1905. The largest flood recorded is 20 200 sec-ft. In January, 1916, when the Gila discharged a flood of 200 000 sec-ft., the flow of the Little Colorado was 16 000 sec-ft. There is a feasible site on the Little Colorado where a dam to raise the water 50 ft. would make possible the regulation of the flow of this stream to 600 sec-ft., or less. Furthermore, a power dam on the Colorado at the mouth of Diamond Creek would smooth out any flood that may reasonably be expected from the Paria, Little Colorado, or Kanab Creek. There is no record of a flood from the Virgin River, that would endanger property on the Lower Colorado.

For a given height of dam at Lees Ferry and at Boulder Canyon, the greater storage capacity may be had at Lees Ferry. In Table 3, Mr. Davis gives, under the heading, "Boulder Canyon Reservoir", a capacity table which applies at the dam site in Black Canyon. The dam site in Black

Canyon is 18 miles below the site in Boulder Canyon. The writer understands that Mr. Davis recommends the construction of a dam at the Boulder Canyon site.

In regard to the loss of water by evaporation, Mr. Davis might have gone one step further in order to show the evaporation that would take place if the Boulder Canyon site were to be developed. Evaporation at Boulder Canyon would be greater than at Lees Ferry for a given area of water surface. With equal storage capacity at Lees Ferry and Boulder Canyon, the loss due to evaporation would be practically the same.

Mr. Davis correctly states that the underground conditions at the Lees Ferry dam site are unknown and that these conditions at Boulder Canyon have been carefully investigated. More than $200 000 has been made available for the investigation of the Lower Colorado, nearly all which has been spent at Boulder and Black Canyons.

As the depth to bed-rock at the Lees Ferry dam site is considerably less than 100 ft., the suggestion made by Messrs. Stevens and Fortier are worthy of further consideration. With bed-rock comparatively near the surface and the condition for unwatering the site favorable, the writer has changed his opinion with reference to the plan for making the rock-fill water-tight. It seems probable that a feasible plan for a combination concrete rock-fill dam can be devised.

Mr. Fortier has submitted valuable data regarding the silt problem on the Lower Colorado, but the writer cannot accept his conclusion that, in order to store a greater quantity of silt, the first flood-control dam should be built at Boulder Canyon.

The constructive criticism submitted by so many engineers is greatly appreciated by the writer, and his objective has been realized. However, he wishes to admit that it was a mistake to present for discussion a proposition to build a rock-fill dam 780 ft. in height above its foundation. The magnitude of the project seemed to stagger the imagination of the engineers who discussed the paper. A more valuable discussion would have resulted if the writer had designed a dam about 400 ft. high. The principles, however, would be the same.

If a thorough study of the physical condition on the Lower Colorado should show that the first flood-control dam should be at Lees Ferry, the time will have arrived for careful study to determine the type of dam most feasible for this site.

No doubt a board of competent engineers would give due consideration to the rock-fill type of dam as well as the concrete dam of the gravity, arch, and multiple-arch type. As the depth to bed-rock at the Lees Ferry dam site is less than 100 ft., some engineers believe that a concrete dam of the multiple-arch type would be safe and also the most economical.

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Paper No. 1513

BOND STRENGTH OF WOOD PILES IN CONCRETE

By R. R. Lundahl,* Assoc. M. Am. Soc. C. E.

Synopsis

For convenience, this paper has been subdivided, as follows:

First.—The conditions are discussed, that necessitated determining the bond strength of wood piles in concrete in connection with the design of the new sewage disposal plant for Milwaukee, Wis.

Second.—A general description of the test specimens used in obtaining the results contained in this paper, together with their preparation, are given.

Third.—A detailed account, description, and discussion of each of the tests as conducted, are given.

Fourth.—The testing apparatus and the methods of testing are explained.

Fifth.—The combined results are discussed, a table of the results of all the tests is given for ease in comparison, and conclusions from the results are drawn.

General

While plans and specifications were being made by the Sewerage Commission of the City of Milwaukee for the new sewage disposal plant, the question of "the bond strength of wood piles in concrete" had to be considered.

The new plant is to be built at the eastern edge of the city, on what is now a peninsula bounded on one side by Lake Michigan and on the other two sides by the Kinnickinnic and Milwaukee Rivers, respectively. The main part of the plant is to be located on made land, within a pile bulkhead and steel sheet-piling coffer-dam, extending 1 000 ft. into Lake Michigan along the south harbor entrance pier. The coffer-dam has been built, and the area

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within it has been dredged to the average elevation of the bottom of the aeration and sedimentation tanks that are to be constructed therein.

It will be necessary to place the tanks on pile foundations. With a 16-ton load on each pile, the dead weight of the tanks and the sewage, when the tanks are full, requires an average pile spacing of 5-ft. centers. The weight of the tanks, when empty, is not sufficient to overcome the buoyancy; therefore, it is necessary to anchor the structure to the pile foundation. With a 5-ft. pile spacing, the uplift on each pile averages 6 tons, and the question arose as to whether the skin friction or bond strength of a plain wooden pile in concrete was sufficient to withstand this uplift. A thorough search for information on this subject was made, but no data applicable to the conditions were found. The engineers of the Sewerage Commission favored the use of a mechanical anchorage of some type, although other engineers believed that such anchorage was unnecessary. A majority, however, seemed to favor the deformed type of mechanical anchorage, and this brought up the question of cost. The cheapest method of deforming a pile-head meant an additional cost of at least $1 per pile. As there are about 24 000 piles to be driven, this would entail a minimum additional cost of $24 000, and more elaborate methods of deformation would run as high as $2 or more per pile. This additional expense was not justified without further investigation, because there was no assurance that the deformed anchorage would serve the purpose any better than the plain pile-head. It was decided, therefore, that a series of tests should be conducted, in order to determine whether any type of mechanical anchorage was necessary, and the writer was entrusted with the carrying out of these tests. Thirteen cedar posts, ranging from 6 to 7½ in. in butt diameter, were selected for test specimens. Cedar was chosen because it was available, and it was thought that if the tests proved satisfactory on a soft wood, harder woods could be used with a considerable factor of safety. The posts which had been cut in the winter of 1919 and 1920 and stored in the open, were selected at random from a stock pile, the only requirement being that they should be straight.

The posts were prepared for thirteen types of anchorage, some of which differed only slightly. These types will be explained as the results of each test are given. The butts of the posts were cut squarely across, so that a fresh area of the ends would be exposed to the concrete, thus giving a condition similar to that when a pile is cut off at grade before it is covered with concrete. The posts were embedded 1 ft. in a slab of concrete 4 ft. square and 1 ft. thick, having a sub-slab, directly under the post, 2 ft. square and 1 ft. thick, cast with the larger slab. The object of the sub-slab was to prevent uneven curing effects around the end of the post and to give a condition similar to that obtained in practice. A 1:2:4 mixture of concrete, with washed sand and crushed gravel as the aggregate, was used. The concrete was mixed in a one-bag mixer, and conveyed in a car to a platform, from whence it was shoveled into the forms by hand. The specimens were made in the Casting Plant of the Sewerage Commission, in March, 1921. The building was heated so that there was no possible danger from freezing. The forms were extended about 6 in. above the slab, in order that the concrete might be cured

under water for 30 days, thus giving a condition similar to that to be expected in subaqueous work.

Test No. 1.—The specimen for this test was 7⅛ in. in diameter at the butt, and 7 in. at the concrete surface. The butt was immersed in water for 7 days before it was concreted. This, in the writer's opinion, was the worst condition that could obtain. The concrete was poured on March 24, 1921, and the specimen was tested on April 27, five weeks later.

In order that any movement of the concrete could be noted, the loading box was blocked up 6 in. above the slab, as shown in Fig. 1. Another box or shield was built around the post so that no frictional resistance would be caused by the material against it. The net weight per vertical foot of load in the box was 2 797 lb., the box weighed 475 lb., and the concrete anchorage 2 960 lb. The loading of the specimen to the capacity of the box was completed about 11:00 A. M., April 27, the total load being 14 760 lb. No movement had been noted at noon, but some time between noon and 2:00 P. M., the specimen failed, and as no one witnessed the failure, the time the load had been sustained, could not be determined.

The bond area of this specimen was 266 sq. in., and the failure occurred under a sustained load, for bond, of 55.5 lb. per sq. in. The results of the other tests showed that, as far as bond strength was concerned, this specimen was tested under the worst condition possible.

Test No. 2.—Specimen No. 2 was similar to Specimen No. 1, except that it was not immersed in water before it was placed in the concrete. The diameter at the butt was 6¾ in. and at the concrete surface, 6⅝ in., and the bond area was 252 sq. in.

The loading was started on May 2, 1921, the specimen having aged 5 weeks and 5 days. In order to determine the exact point of failure, the load was applied as shown in Table 1.

TABLE 1.

Load, in pounds.	Time.	Date.	Hours, load was sustained.
7 200	9:00 A.M.	May 2, 1921	6
8 300	3:00 P.M.	" 2, "	23
9 700	2:00 P.M.	" 3, "	21
10 200	11:00 A.M.	" 4, "	5
11 000	4:00 P.M.	" 4, "	19
11 500	11:00 A.M.	" 5, "	5
12 000	4:00 P.M.	" 5, "	43
13 400	11:00 A.M.	" 7, "	21
12 500	8:00 A.M.	" 8, "	31
13 700	3:00 P.M.	" 9, "	22
14 100	1:00 P.M.	" 10, "	23
14 800	12 00 Noon	" 11, "	48
19 500	12:00 Noon	" 13, "	1½
21 000	1:30 P.M.	" 13, "	18½

At 21 000 lb., the capacity of the crane had been reached, and this load was allowed to remain until 8:00 A. M., May 14, when the specimen was lowered and the load removed. A bond strength of 83.3 lb. per sq. in. had been developed with no sign of failure.

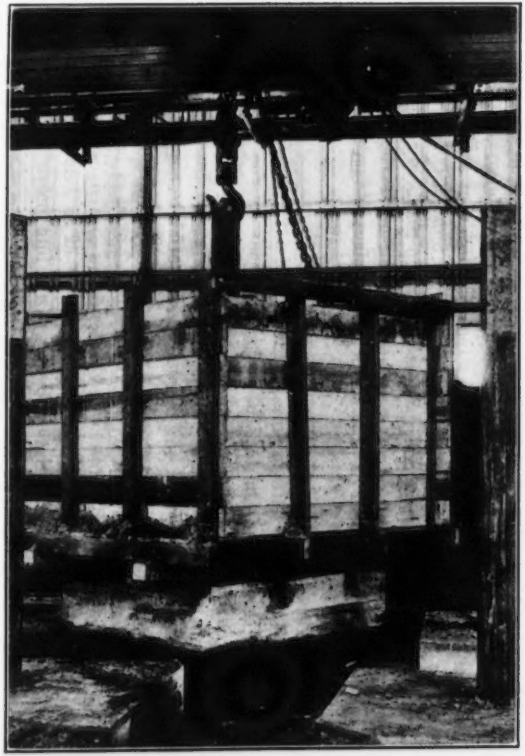


FIG. 1.—LOADING BOX BLOCKED UP 6 IN. ABOVE THE SLAB FOR TESTING.

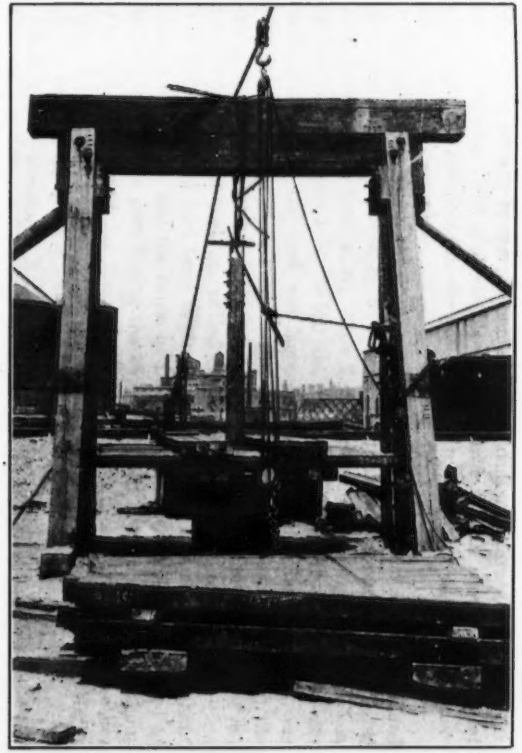


FIG. 2.—VIEW OF TESTING APPARATUS.

It was decided to move all the specimens to the open, build a gallows-frame strong enough to carry a much heavier loading, and use billet steel for loading purposes. The concrete blocks were covered with about 6 in. of wet sand to prevent shrinkage in the posts. In spite of this precaution, considerable shrinkage took place.

The test of Specimen No. 2 was resumed on June 1, 1921. A 10 200-lb. load was placed on the specimen by 5:00 P. M., and allowed to remain over night. On examination at 8:00 A. M., June 2, a slight cleavage was noted. When the load was increased to 13 000 lb., the post pulled from the concrete about $\frac{1}{8}$ in. When the load was increased to 16 000 lb., the movement was about $\frac{1}{4}$ in., but it was so gradual that it could not be observed during the loading. The load was then increased, in 360-lb. increments, to 19 500 lb., at which loading the post was pulled entirely from the concrete.

A 21 000-lb. load had been applied previously on this specimen without any sign of failure, and the cause of failure at a lesser load is attributed to the shrinkage of the post during the 18 days it had been in the open without being kept wet, except for the moisture in the sand covering. The weather during this period was exceptionally warm most of the time, and the slight rainfall drained off quickly.

Test No. 3.—Specimen No. 3 was immersed in water for 7 days before it was concreted. The diameter at the butt was $7\frac{7}{8}$ in. and, at the concrete line, $7\frac{3}{8}$ in. The concrete had aged 71 days when the loading of the specimen was started on June 2. The load was placed in 360-lb. increments and totaled 17 400 lb., at 6:30 P. M. No failure could be noted, and the load was allowed to remain until noon the following day, when it was observed that the post had pulled $\frac{1}{8}$ in. from the slab. The failure, under a sustained load of 17 400 lb., was equivalent to a bond strength of 60.6 lb. per sq. in. The load was increased to 20 300 lb., and the post pulled gradually to $\frac{1}{4}$ in., which was considered sufficient evidence of failure, and further operation was discontinued.

To guard against the conditions in Test No. 2, the remaining test specimens were thoroughly soaked, and the result of Test No. 3 shows that it was helpful.

Test No. 4.—Specimen No. 4 was dry when the concrete was poured and had aged 72 days when the loading was started at 3:30 P. M., on June 3. The diameter of the post at the butt was $7\frac{5}{8}$ in. and, at the concrete line, $7\frac{1}{4}$ in. A 16 000-lb. load had been applied by 8:30 P. M., with no indications of failure. This load was allowed to remain until 10:00 A. M., June 4, $13\frac{1}{2}$ hours, but no signs of failure were noted. The load was increased to 19 520 lb. and allowed to remain until noon, June 6, at which time the bond strength appeared to have failed slightly, but there was some doubt as to this fact. The load, therefore, was increased to 23 840 lb., at which a slight movement was noted, and, at 25 640 lb., the post had been pulled $\frac{3}{4}$ in. The specimen was now unloaded, and it was settled definitely that failure had occurred at 19 520 lb. which was equivalent to a bond strength of 69.7 lb. per sq. in.

Test No. 5.—Specimen No. 5 was 7 in. in diameter at the butt and $6\frac{1}{8}$ in. at the concrete line. The concrete was banked up about 4 in. around the post in the form of a cone. The idea was that the area of concrete in con-

tact with the pile would be increased at little added expense. This method was considered to be the next cheapest, if the results of the tests on the plain anchorages proved to be inadequate.

This specimen had aged 75 days when the loading was started at 7:45 P. M., on June 6. At 8:30 P. M., a load of 10 960 lb. had been applied, which was allowed to remain until noon on June 7, with no sign of failure. The load was then increased to 25 640 lb. by 8 000, 5 000, and 2 000-lb. increments, respectively, and at 7:00 P. M., the first sign of failure was noted. Further loading to 28 160 lb. resulted in the failure of the cedar post in tension.

If the embedded depth of the post is taken as 12 in., a bond strength of 100 lb. per sq. in. was developed. If the embedded area of the post is used, the bond strength was 75.8 lb. per sq. in. The cedar post failed in tension under a unit stress of about 850 lb. per sq. in.

Test No. 6.—Specimen No. 6 was of the same type as Specimen No. 5, but had a slightly larger butt diameter. The concrete had aged 79 days when the loading was started at 9:00 A. M. At 11:00 A. M., a load of 16 200 lb. had been applied when the first sign of failure was noted. A bond strength of only 60.0 lb. per sq. in. had been developed.

The load was increased to 23 120 lb., at which the post pulled from the concrete. The post was found to be sound, but there had been no bond to the bottom of it, whereas, in the other tests, this was not the case. The area of the bottom of the post was about 33 sq. in., and the bond on this area would have had a material affect on the total load sustained.

In the foregoing tests, the unit stresses have been calculated from the areas of the sides alone, but the bond to the bottoms undoubtedly plays an important part. The exact relation could not be determined except by tests.

When Specimen No. 6 was concreted, the post settled so that it was embedded more than 12 in. in the concrete. Accordingly, it was pulled up to the 12-in. mark. In doing this, the bond to the bottom of the post must have been destroyed. The results seem to indicate that the bond to the end fibers plays a more important part than that to the sides of the post.

Test Specimens Nos. 1 to 6

Tests Nos. 1 to 6 complete all those without any type of deformed anchorage. The conclusions from these tests are that the enlargement of the pile-head has no effect on the load sustained until the bond strength has failed. In every test, it was noted that, when the bond failed, the concrete usually dropped suddenly about $\frac{1}{8}$ in., and then continued to move slowly to $\frac{1}{4}$ in. Between the $\frac{1}{8}$ and the $\frac{1}{4}$-in. movement, the enlargement of the head became effective. In nearly every case, after this point had been reached, the load was applied without additional failure until the post either broke or pulled from the concrete.

The writer believes that as long as the concrete adheres to the bottom of the post, there is no chance for the fibers to compress. This is probably the cause of the sudden drop as soon as the bond has failed. The load necessary to overcome the bond was sufficient to compress the fibers enough to allow the concrete to drop the $\frac{1}{8}$ in. After this, an additional load was necessary to compress the fibers sufficiently to allow the post to be pulled even slightly.

A movement of $\frac{1}{8}$ in. might be injurious to a structure, and where it is necessary for the piles to withstand any upward pressure, it is unsafe, therefore, to consider anything but the bond strength of the concrete to the wood pile. It would seem safe to consider the bond strength of the concrete to the wood pile to be 50 lb. per sq. in., which value is less than any of the results obtained, but the entire static head should be used in designing, in order to obtain an additional factor of safety.

Whether a pile of larger diameter than those used would develop a similar bond strength can be determined only by tests. The Sewerage Commission intends to conduct a series of such tests, in order to secure this information.

Tests on the Deformed Specimens

Test No. 7.—Specimen No. 7 had an average butt diameter of $6\frac{3}{4}$ in. and no enlargement. Four 3-in. track spikes were driven, 90° apart, into the butt, the heads extending about 1 in. for anchorage. The concrete in this specimen had aged 80 days, and the test was started at 8:30 A. M., June 10, 1921. At 11:00 A. M., after a load of 16 640 lb. had been applied, the first sign of failure was noted, the post having pulled about $\frac{1}{8}$ in. The load was increased to 17 360 lb., and as the post continued to pull from the concrete, the test was stopped. The bond area on the sides was 257 sq. in., and the bond stress developed was 64.7 lb. per sq. in.

Test No. 8.—This specimen had a diameter of $7\frac{1}{16}$ in. and an enlargement of $\frac{7}{16}$ in. A 1-in. pin, 30 in. long, was passed through a 1-in. hole drilled through the post 6 in. from the bottom. About 1 ft. of the pin extended from each side of the post. The concrete had aged 81 days when the loading was stated at 1:45 P. M., June 10. At 4:00 P. M., a load of 18 080 lb. had been applied and no signs of failure had been noted. This load was allowed to remain until 8 A. M., June 11, when it was increased to 22 040 lb., under which, at 8:50 A. M., the first sign of failure was observed. The load was increased to 22 400 lb. and the post pulled about $\frac{1}{8}$ in., which load is equivalent to 84.4 lb. per sq. in. on the bond area.

The test was stopped, as it had already failed beyond the point of safety. No doubt, the pin through the post increased the load that could be sustained by this specimen. After the failure of the bond, which probably occurred before the line of fracture was noted, the specimen held until the fibers in contact with the pin had compressed. When this occurred, the post pulled from the concrete, and the line of failure was noted.

Test No. 9.—Specimen No. 9 had an average butt diameter of $7\frac{1}{8}$ in., and 1 ft. from the butt, it was sawed on two opposite sides to a depth of 1 in. The wedges thus formed, tapering to 1 in. at the concrete line, were then chopped out, thus making the post $7\frac{1}{8}$ in. in diameter at the butt, and, at the concrete line, a flattened ellipse, 5 by 7 in.

The concrete had aged 81 days when the loading was started at 11:30 A. M., and by 2:00 P. M., a load of 16 280 lb. had been applied when the first sign of failure was noted. The bond strength developed was 61.2 lb. per sq. in. The load was increased as follows: 17 360 lb., $\frac{1}{4}$-in. movement; 18 080 lb., $\frac{3}{8}$-in.

movement; and 18 520 lb., 1-in. movement. At 22 400 lb., the post pulled from the concrete. Examination disclosed that there had been no shearing action; the fibers had been compressed and pulled through the 5-in. hole in the concrete.

The preparation of piles in this manner would be expensive, and the results of the test show that this added expense would be unwarranted.

Test No. 10.—This specimen was 6⅝ in. in diameter and had been expanded by driving a 1 by 12 by 6½-in., hard-wood wedge flush with the butt, thus making the bottom of the post elliptical in shape, 6⅝ by 7⅝ in. Before the wedge was driven, a band was placed around the post at the concrete surface, which band was not removed during the test. This specimen was 83 days old. The load was started at 8:45 A. M., and by 11:15 A. M., the first sign of failure was noted with a load of 15 920 lb., which is equivalent to a bond strength of 63.7 lb. per sq. in. Under a load of 18 800 lb., the post had pulled ¼ in., and ⅜ in. with 20 240 lb. It was seen that this method did not increase the bond strength, and the loading was not continued.

Test No. 11.—Specimen No. 11 was trimmed to a wedge shape on four sides, instead of two sides, as was done with Specimen No. 9. The average diameter of the butt was 6½ in. and, at the concrete line, the post was about 4½ in. square.

The concrete in this specimen had aged 84 days and the first sign of failure was noted with a load of 13 760 lb. With a bond area of about 244 sq. in., this load was equivalent to 56.4 lb. per sq. in. From this result, it is obvious that the trimming of a pile-head to this shape would not be justified. When the load was increased to 18 080 lb., the post pulled from the concrete about ¼ in., and at 22 400 lb., it pulled ½ in. After remaining for 2 hours without further movement, the load was increased to 23 840 lb., at which the post parted at the concrete line, under a load of 1 180 lb. per sq. in.

Test No. 12.—Specimen No. 12 was prepared by driving two wedges in the butt at right angles to each other. An iron band was placed around the post, 1 ft. from the butt, before the wedges were driven. The band was used to give the same results as would be obtained from the soil around the pile-head if the wedges were driven in the field, after the piles were in place. On this specimen, the band was removed before the test was started.

This test was begun on June 14, at 3:00 P. M., and was continued, with 360-lb. increments, until noon, on June 15, when the first sign of failure was noted with a load of 15 200 lb. The average diameter at the butt was 6⅜ in., and the bond area, disregarding the increase in size due to the wedges, was 240 sq. in., thus giving a unit bond strength of 63.3 lb. per sq. in.

In Test No. 10, with one wedge, the unit bond strength was 63.7 lb. These results would indicate that there would be no reason for using more than one wedge. This method of deformation would be considerably cheaper than trimming the post to a wedge shape, and also gives a higher unit strength, but there is no indication that the bond strength would be any greater than that in an anchorage without any deformation.

The driving of wedges in the pile-heads would not injure the original surface fibers, as would be the case if the methods in Tests Nos. 9 and 11 were

followed; but there would always be some "give" between the post and the wedges, and, as the load was applied, this "give" would cause the bond to break sooner than in a solid post.

Test No. 13.—In Specimen No. 13, two 1-in. pins at right angles to each other had been driven through holes in the post, 6 in. from the butt. The pins extended 1 ft. into the concrete, on four sides.

The post had a slight butt enlargement, an average diameter of 6½ in., and a bond area of 240 sq. in., after deducting for the area of the pins. The loading was started on June 16 at 8:00 A. M., and by 11:00 A. M., a load of 15 920 lb. had been applied, when failure was noted. This load was allowed to remain, but no further failure occurred. A bond strength of 66.3 lb. per sq. in. had been developed, but the failure had been so slight that the movement would not have been injurious to any structure. In fact, there was a question as to whether the slight fracture noted was really a failure due to the pulling out of the post, or whether it was due to the elongation of the wood fibers at the concrete line. The latter cause would seem to be the more plausible, because, under continued loading, no further movement was noted.

Under a load of 20 600 lb., the post parted, and the movement at the concrete line was found to be small. A bond strength of about 86 lb. per sq. in. had been developed without an injurious amount of movement.

The pin types of deformation are by far the cheapest that were tried, and the results indicate that only one pin would be necessary, if it was decided that a deformed anchorage was needed.

Description of Testing Apparatus and the Method of Testing

Fig. 2 shows the testing apparatus. The gallows-frame was designed to carry a load of 40 tons.

The method of testing was as follows: A locomotive crane would swing the specimen under the frame so that the pin could be inserted through the sling and the shackle-plates, the sling being of such length that the specimen was clear of the ground when it was released by the crane. The steel billets, each of which weighed 360 lb., were handled by a block and tackle running on a trolley. They were lowered with the least possible jar to the specimen and were racked so that a clear space was left around the post for observation and as a guard against friction of the load on the post. The post and the concrete adjacent to it were thoroughly chalked before loading was started, in order that any movement could be better detected. The increments of load were applied slowly, and as the point of failure was reached, the specimen was closely watched for any sign of such failure. As soon as the slightest line appeared, the loading was stopped for a time, in order to allow the post to pull as much as possible under that load. The points of failure were determined closely from the results of the test on Specimen No. 2 which was loaded in small increments and allowed to stand for several hours before additional load was applied.

Conclusions

Table 2 gives the results of the tests and affords an opportunity for comparing the different types of anchorage tested, for drawing conclusions as to

TABLE 2.

Test no.	Type of anchorage.	Diameter at butt, in inches.	Diameter at concrete line, in inches.	Enlargement, in inches.	Total load sustained, in pounds.	Area, in square inches of sides in bond.	Area of butt, in square inches.	Bond strength (sides), in pounds per square inch.	Bond strength, sides and bottom, in pounds per square inch.	Remarks
1	No deformation	7⅛	7	⅛	14 760	266	39.9	55.5	48.2	Pile butt soaked for 7 days before being concreted.
2	"	6¾	6⅞	⅛	21 000	252	35.8	83.3	72.9 cu. ft. dry	
3	"	7⅞	7⅞	½	17 400	327	48.7	60.6	51.8	Pile butt soaked as in Test No. 1.
4	"	7⅞	7¼	⅜	19 520	280	45.6	69.7	59.9	Butt dry.
5	Concrete banked up about 4 in	7	6 11/16	5/16	25 640	258-338	38.5	100-75.8	68.1	Post pulled apart at 28 160 lb.
6	Concrete banked up about 4 in	7½	7⅛	⅜	16 280	276	44.2	59.0	.....	No bond to the bottom.
7	Four small track spikes driven	6⅞	6¾	⅛	16 640	257	37.1	64.7	56.6	Total movement ¼ in. at a load of 17 860 lb.
8	1-in. pin through butt one way	7 1/16	6¾	5/16	22 040	260	39.0	84.4	74.0	Total movement ⅛ in., load, 22 400 lb.
9	Trimmed to a wedge shape on two sides	7⅛	7	⅛	16 280	266	39.9	61.2	53.2	Post pulled out, load, 22 400 lb.
10	Expanded 1 in. by wedge driven into butt	6⅝	6⅝	None	15 920	250	34.5	63.7	55.9	Total movement ⅜ in., load, 20 240 lb.
11	Butt trimmed to wedge shape, four sides	6⅝	6⅜	¼	13 760	244	34.5	56.4	49.4	Post broke at top of concrete, load, 23 840 lb.
12	Butt expanded two ways by two wedges	6⅜	6⅜	None	15 200	240	31.9	63.3	55.9	Post pulled out, load, 25 280 lb.
13	Two 1-in. pins at right angles through butt	6⅝	6⅜	¼	15 920	245	34.5	66.3	57.0	Post broke at load of 20 600 lb. Movement at breaking point very slight.

the best types, and for determining a safe average bond strength to be used in designs where the factor of bond strength is necessary.

The average bond strength of nine of the tests, not including those immersed in water and those with the pins, is 66.3 lb. per sq. in., when the side area only is considered. Using the combined area of the side and bottom, the average is 58.9 lb. per sq. in. For the piles without deformed anchorage, that is, the first seven, not including the specimens that had been immersed, which are obviously low, but including the test with the spikes, the average strength is 70.5 lb. per sq. in., whereas the average of the deformed anchorages is only 65.9 lb. per sq. in. These results indicate that deformation of the pile-heads has a tendency to decrease the bond strength.

The average strength of the two plain anchorages, Tests Nos. 2 and 4, is 76.5 lb. per sq. in., whereas the average of the two highest tests of deformed heads, the two with pins, is 75.3 lb. per sq. in. Therefore, the results shown by the two best deformed types are not as good as those found with the two plain anchorages.

In conclusion, it may be stated that only the bond strength of the wood to the concrete should be considered. The deformation of the pile-heads does not warrant the added expenditure it will entail. If the substance to be anchored is of metal, a deformed anchorage will be beneficial.

The writer believes that the Sewerage Commission will not consider the use of deformed pile-heads, but will rely on the bond of the concrete to the pile. The piles to be driven will have an average butt diameter of at least 12 in. and are to be embedded to a depth of 10 in. in the concrete. The side area, in bond, is 377 sq. in., and with 12 000 lb. of uplift per pile, a bond strength of 31.9 lb. per sq. in. is necessary. This is less than the lowest result obtained in any of the tests.

The work that brought about the foregoing tests is being done for the Sewerage Commission of the City of Milwaukee under the direction of T. Chalkley Hatton, M. Am. Soc. C. E., as Chief Engineer, with James L. Ferebee, M. Am. Soc. C. E., as Principal Assistant Engineer.

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Paper No. 1514

EXPERIMENTS WITH MODELS OF THE GILBOA DAM AND SPILLWAY*

By R. W. Gausmann,† and C. M. Madden,‡ Associate Members, Am. Soc. C. E.

With Discussion by Messrs. George G. Honness, B. F. Groat, Robert Fletcher, I. Gutmann, Robert F. Ewald, Thaddeus Merriman, and R. W. Gausmann and C. M. Madden.

Synopsis

This paper presents the results of experiments made with models to determine the carrying capacity of a projected spillway channel and a satisfactory section of a stepped, overfall dam. There is also a discussion of the application of the principles of homology to the models in question.

General

The Gilboa Dam, in Gilboa, Schoharie County, N. Y., forms the Schoharie Reservoir which is being constructed by the Board of Water Supply of New York City as a part of the Catskill System. A masonry section, approximately 1 324 ft. long, and an earth dike, about 1 000 ft. long, constitute the dam. The down-stream side of the masonry sections is to be formed of large steps, generally about 20 ft. high. The masonry part of the dam has a maximum height of 160 ft., and it is believed that this is the highest overfall dam of this section ever constructed. This part of the dam will be built of Cyclopean masonry, faced with native gray sandstone.

* Presented at the meeting of October 4, 1922.

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Outline

This paper has been divided into three general headings:

1.—A brief history of the work done and the results obtained.

2.—A description of the experiments in the channel and on the section of the dam.

3.—A study of the differences in the results obtained with models of various scales.

History

The first model of the Gilboa Dam and Spillway was built to determine whether the channel, as designed and shown on the contract drawings under which the work is being done, was ample to carry an assumed maximum flow. The model, built on a scale of 1:50, indicated that the design was adequate, but it also showed that the steps in the overfall section did not break up the falling water as had been intended. Various modifications of the section were made, until one was found (Section 16), that seemed to meet all requirements.

Before accepting this section, it was thought advisable to experiment on a model of larger scale, as minute discrepancies made a considerable difference in the flow. The new model which consisted of only a part of the overfall section, was built on a scale of 1:20. Observation and careful measurement showed a dissimilarity in behavior in these two models, and, therefore, a larger model, on a scale of 1:8, was constructed and tested. The results of these tests proved that, as far as this section was concerned, the behavior of equivalent heads over models of different scale was not identical. This conclusion was checked further by conducting a series of experiments over flat-crested weirs built on three different scales. However, as all the results bore a certain degree of similarity, it was decided to continue the search for a suitable section, but on a model built on a scale of 1:20.

Blocks which later were modified to vanes, were placed on the upper steps to reduce the velocity, and many changes were made in the section, until a form was obtained that met all requirements. This was then reproduced on scales of 1:50 and 1:8. Measurements made over these three models still showed considerable discrepancy, but not as much as had been found in the models without vanes. This section, shown on Fig. 6, has been adopted for the Gilboa Dam to Elevation 1 020.

Experiments

Fig. 1 shows the location of the models, the details of the dam, and the controlling features. Various types of gauges were used to measure the head. Those used for all the later experiments may be seen in Fig. 2, which also shows models of Section 100 on scales of 1:50, 1:20, and 1:8, respectively, all under a proportionate head of 3 ft.

The device used to locate the outer and, in some cases, the inner edge of the jet has been called a "rectometer".* It consists essentially of two scales running in guides that are normal to each other. To determine the vacuum in various parts of the flow, a gauge was made, consisting of a U-tube of glass

with one end open and the other end connected by a rubber tube to a piece of brass tubing of small diameter, which was inserted in the water. This gauge was graduated to read directly to 0.01 lb.

Flood Discharge and Head.—To determine reasonable heads under which to conduct these experiments, the probable maximum flow of Schoharie Creek at Gilboa must be estimated. These data are given in Table 1.

TABLE 1.

River.	Location.	Conditions.	Area of water-shed, in square miles.	Flow, in cubic feet per second.	Flow, in cubic feet per second per square mile.	Authority.*
Schoharie Creek..	Gilboa, N. Y.....	Occasional flood.	314	34 850	111	(1)
" " ..	" "	Rare flood.....	314	60 730	193	(2)
" " ..	Prattsville, N. Y.....	Flood of 1903....	235	28 000	119	(3)
Pequannock.....	Mecopin Intake, N. J..	" " "	62	6 000	90.8	(4)
Conemaugh	Johnstown, Pa.....	" " 1889....	48.6	10 450	215	(5)
"	" "	Occasional flood.	48.6	10 790	222	(6)
"	" "	Rare flood.....	48.6	15 115	311	(7)
Calaveras	California	Flood of 1911.....	395	69 520	176	(8)
Devil's Creek.....	Iowa.....	" " 1905.....	143	185 900	1 300	(9)
Great River.....	Westfield, Mass.....	.....	350	53 170	151.9	(10)
Schoharie Creek..	Gilboa, N. Y.....	Flood of 1869.....	314	44 100	140	(11)

* (1), (2), (6), and (7), computed from formula of the late Emil Kuichling, M. Am. Soc. C. E., found on p. 844, Report of the State Engineer on the Barge Canal, 1901.
(3), *Water Supply and Irrigation Paper 97*, 1903.
(4), Supplement to Report of the Northern New Jersey Flood Commission, March 4, 1904.
(5), *Transactions*, Am. Soc. C. E., Vol. XXIV (1891), p. 431.
(8) and (9), Report of Arthur E. Morgan, M. Am. Soc. C. E., Chief Engineer, Miami Conservancy District, Vol. I, p. 77.
(10), "American Civil Engineers' Pocketbook", Second Edition, p. 905.
(11), Based on flood of 1903 and gauge height and resulting cross-section of the stream for the flood of 1869.

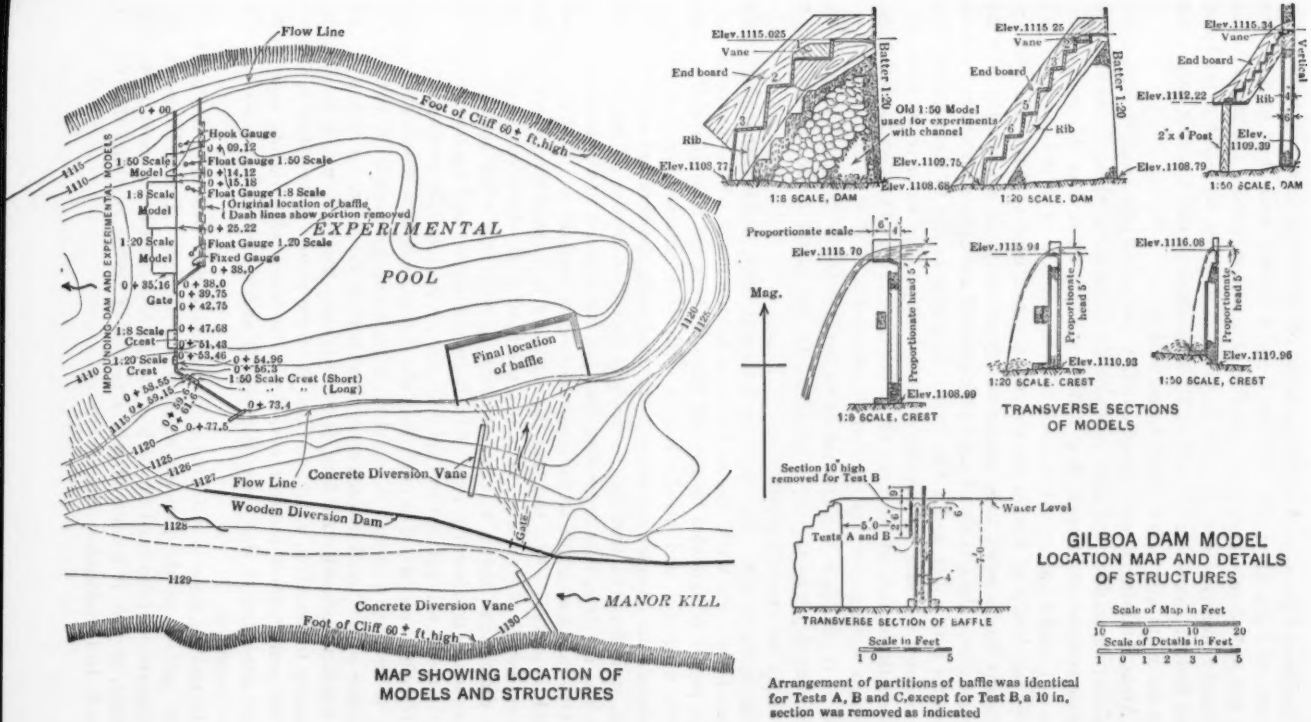
A study of Table 1 made it evident that a flood of 160 cu. ft. per sec. per sq. mile might occur. Table 2, computed from the formula,* $Q = C\,L\,(H)^{\frac{3}{2}}$, in which $C = 2.64$ and $L = 1324$ ft., gives the head for various flows over the dam.

TABLE 2.

Head, in feet.	Discharge, in cubic feet per second.	Discharge, in cubic feet per second per square mile.
1	3 495	11
2	9 884	31
3	18 160	57
4	27 960	90
5	39 074	130
6	51 381	163
7	64 727	206
8	79 070	251
9	94 365	301
10	110 512	352

It was thought best to conduct experiments with heads up to 10 ft., as the maximum flood flow is indeterminate and as the vanes near the crest may cause a retardation of velocity.

* *Water Supply Paper No. 200*, U. S. Geological Survey.



Arrangement of partitions of baffle was identical for Tests A, B and C, except for Test B, a 10 in. section was removed as indicated

FIG. 1.

Spillway Channel.—A concrete model of the entire dam and spillway, on a scale of 1:50, was built and tested under a head equivalent to 6 ft. over the full-scale dam. Assuming a similarity in action between the model and its full-scale prototype and that the head selected would represent a maximum flood flow, it seemed to be evident that the channel was not only ample, but that its width and depth might be reduced.

After obtaining a satisfactory overfall section on this small scale, another model of the spillway and dam, in which the width of the channel was modified, was constructed and tested. Fig. 3 shows a head of 5 ft. over this model (Section 16), under an equivalent head of 5 ft. and flow of 40 000 sec-ft. Fig. 4 shows the depths of flow in the channel due to a head of 6 ft. The contours represent depths which were approximately constant. In general, the high and the low places along the wall and in the channel, are approximately in the same place for both a 3-ft. and a 6-ft. head. This may be due to the contraction in the channel opposite the beginning of each step. The water is the same general depth throughout the channel for the same head. Thus, on the 6-ft. head, the high water at the upper end of the channel is 13 ft. deep and, at the lower end, 14 ft., which is the greatest depth for this head. With a 3-ft. head, the maximum depth at the upper end is 5.7 ft. and at the lower end, 5.9 ft., and 6.7 ft. is the greatest depth.

As much of this work was done during freezing weather, the spray would congeal above the water line and leave a definite profile of the maximum flow. This profile is indicated by the full lines on both the retaining wall and the north wing-wall and checks closely with that obtained by measurement. Although the channel has never been tried with the accepted section of the dam, it is believed that the results will be better, as the falling jet will be closer to the dam, thus increasing the available capacity of the channel.

One might conclude from these experiments that the width of the channel could be reduced 50 ft. and that the retaining wall could be reduced from its present height of from 19 to 24 ft. to a height of 17 ft. However, no changes in the dimensions shown on the contract drawing are contemplated, largely because the maximum possible flow is indeterminate and because the action on the model will not be identical with that on its full-scale prototype.

Overfall Section.—The purpose of the experiments with the overfall section was to develop a profile in which a fairly equal proportion of the jet will impinge on the tread of each step, thus causing the jet to be broken up and its velocity reduced. At first, it was thought that this might be done by changing the location, width, and height of the steps. A considerable number of the combinations possible were built, on which experiments were made. Those experiments that gave fair results and some that gave poor results were measured.

In making these modifications, it was found that reasoning was comparatively useless, as the problem was too complicated. Minor changes in the section greatly influenced the path of the jet and often in the most unexpected manner, so that it became largely a matter of trying anything that did not seem to be too absurd.

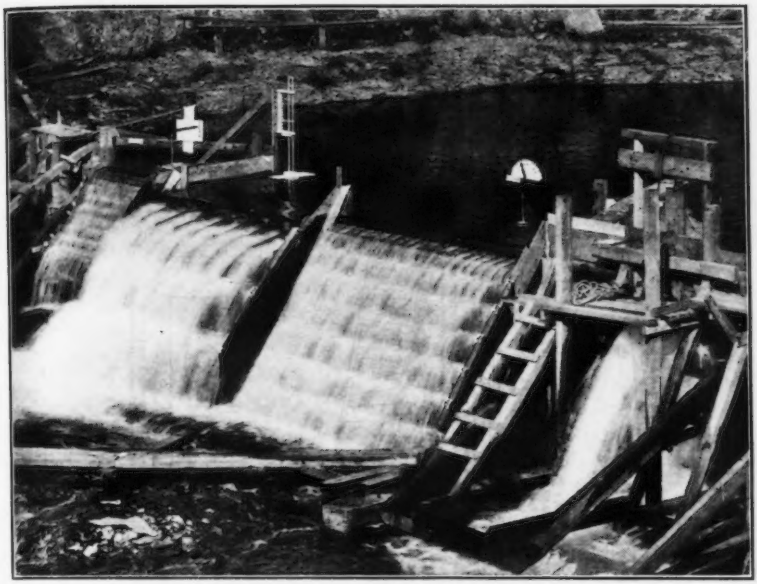


FIG. 2.—MODELS OF SECTION 100, GILBOA DAM, ON SCALES OF 1:50, 1:20, AND 1:8, RESPECTIVELY.

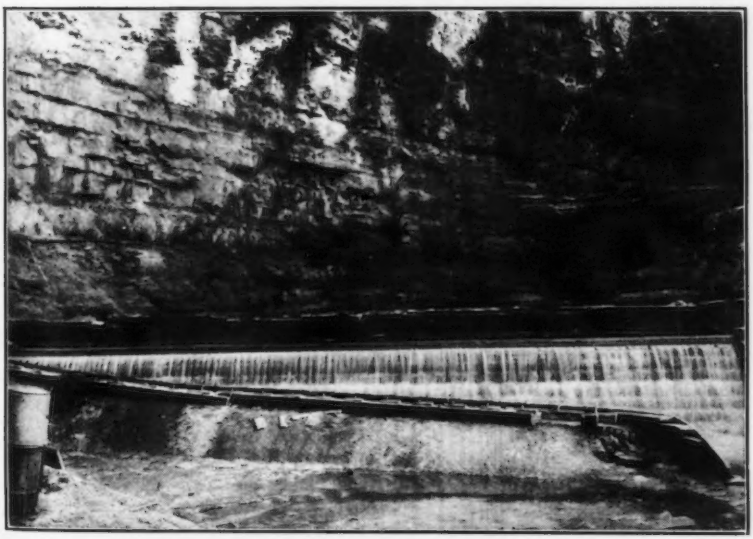


FIG. 3.—MODEL OF SECTION 16, GILBOA DAM, SHOWING EQUIVALENT HEAD OF 5 FT.





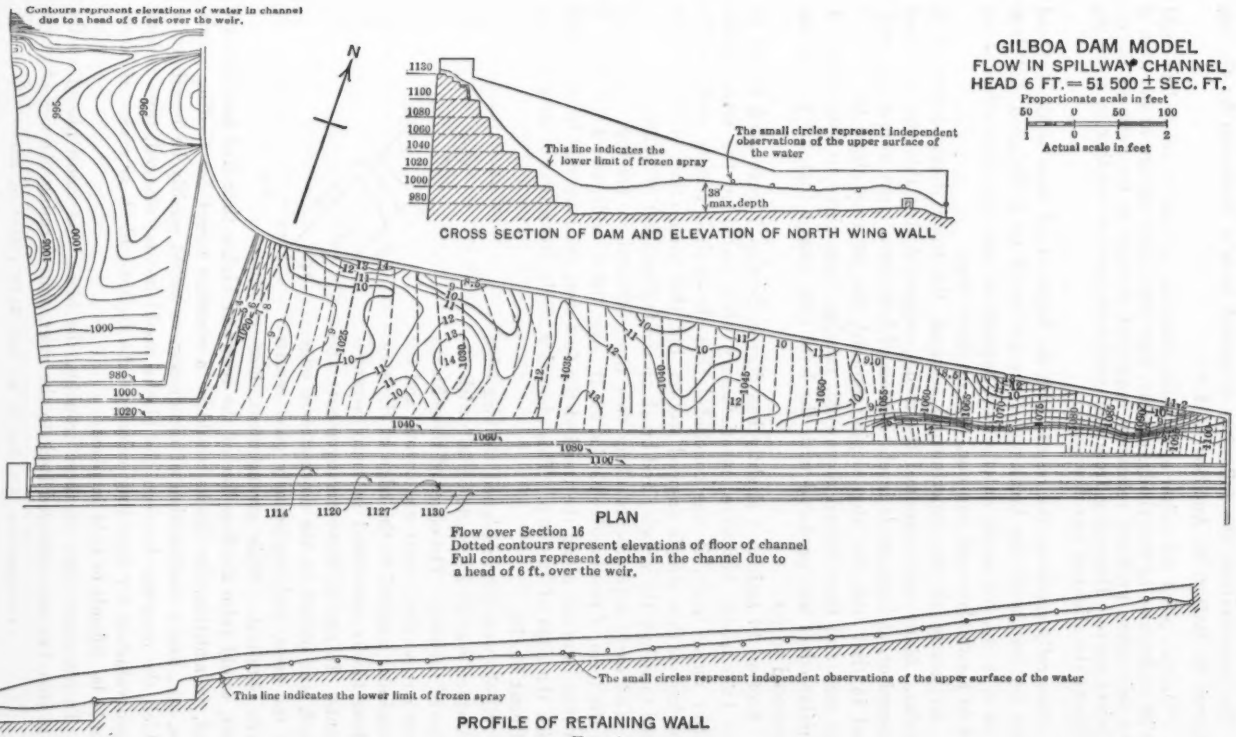

Contours represent elevations of water in channel due to a head of 6 feet over the weir.
N
GILBOA DAM MODEL
FLOW IN SPILLWAY CHANNEL
HEAD 6 FT. = 51 500 ± SEC. FT.
Proportionate scale in feet
50 0 50 100
1 0 1 2
Actual scale in feet
The small circles represent independent observations of the upper surface of the water
This line indicates the lower limit of frozen spray
35' max. depth
CROSS SECTION OF DAM AND ELEVATION OF NORTH WING WALL
PLAN
Flow over Section 16
Dotted contours represent elevations of floor of channel
Full contours represent depths in the channel due to a head of 6 ft. over the weir.
The small circles represent independent observations of the upper surface of the water
This line indicates the lower limit of frozen spray
PROFILE OF RETAINING WALL

FIG. 4.

In corroboration, the following is quoted from a discussion by the late George W. Rafter, M. Am. Soc. C. E.*:

"On this point the writer again calls attention to what seems to him ought to be understood by everybody, namely, that very slight change in the form of the weir often produces relatively important changes in the form of the coefficient curve. Reasoning from one coefficient curve to another is, therefore, without the slightest significance."

Generally speaking, the results with the better of all these sections showed that for low heads the break-up of the jet would be fairly good, but from 3 or 4 ft. up, too much water would impinge on one step, thus allowing the jet to accumulate and leap one, two, or three steps.

In nearly all the experimental sections, the tread slopes downward or drains. Many who examined the model, suggested that this step be sloped upward or, at least, made level. Models of both these types were constructed, and in that with the step sloping upward, the jet rose above the level of the step and then curved downward, missing from three to four steps. A similar effect was produced with the level step, except that the jet was not projected so far.

Section 16 and, to a slightly less degree, Section 7 gave good results on the 1:50 model, but were unsatisfactory when tested on the larger scales. After many trials, it was concluded that the desired results could not be obtained with a simple stepped section, but that some other means must be used to reduce the velocity and break up the falling sheet of water.

The early experiments along this line consisted in placing obstructions in the form of rectangular blocks, 6.0 ft. long, 3.0 ft. wide, and 4.0 ft. high, at various intervals along the tread of the first step, the long side being parallel with the edge of the step. This showed a slight improvement over sections without blocks.

Blocks were then designed, the long edge of which was at an angle of 30° with the edge of the step. The edges of these blocks were cut, so that a part was normal and the remainder parallel to the edge of the step. These blocks were tried in many arrangements. In one of these arrangements, the block was placed on the down-stream edge of the crest, and particularly good results were obtained. This arrangement, however, would raise the flow line of the reservoir higher than had been planned and, in addition, the blocks would be a constant target for floating débris, ice, etc., and a combination of circumstances might readily be imagined, in which the openings between the blocks would become clogged, thus making them valueless for retarding velocity. An isolated block or series of blocks would be difficult to hold in place.

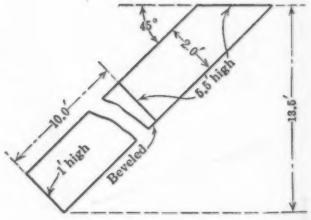


FIG. 5.

To overcome some of these troubles, the block was again placed on the first step, but was extended back to the riser, thus forming a series of vanes.

* *Transactions*, Am. Soc. C. E., Vol. XLIV (1900), p. 391.

The first experiments with these vanes were discouraging, but after many trials, a vane, shown in Fig. 5, was found, that gave better results than the blocks.

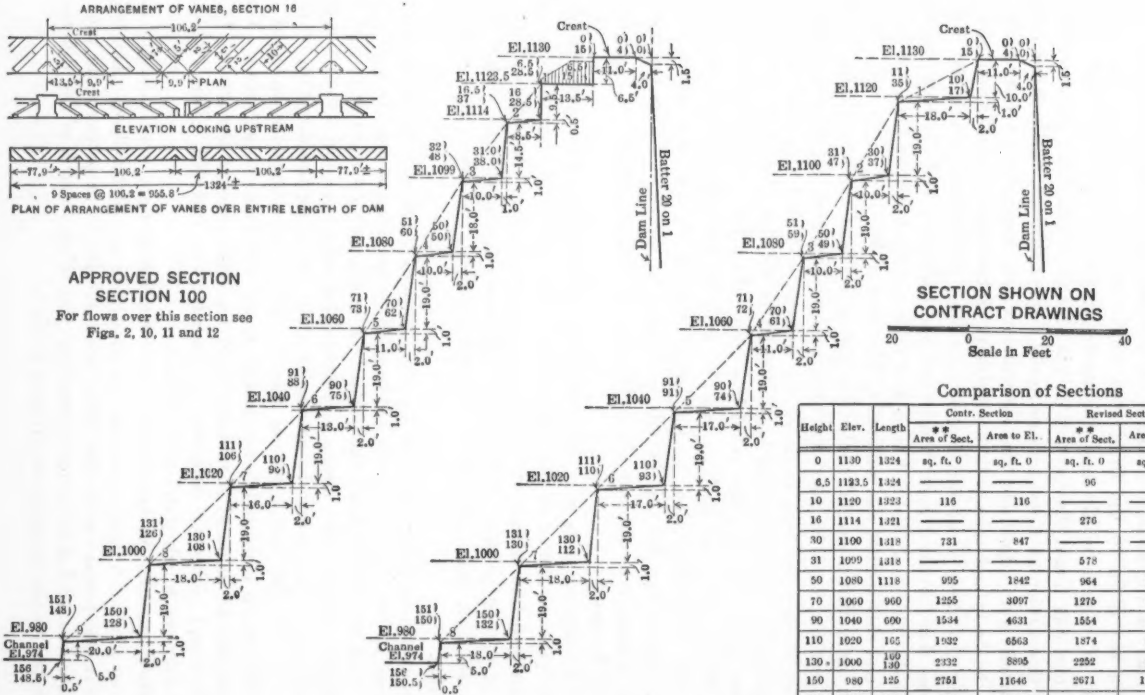
It was decided to accept the crest, first step, and vanes, and to adjust the remainder of the section to them. The model at this time consisted of a wooden top down to the third step, and concrete steps below. By using boards of various widths and thicknesses, many modifications were made, until one was found that seemed to give better results than all the others. With this section as a basis, a new model was constructed, in which the risers and treads were adjustable, and experiments were conducted by starting first with the basic and then with a maximum and minimum section and determining the three most satisfactory sections. Experiments with the three sections thus obtained were then re-run and compared, and the best one was accepted. This was modified later and improved by lowering the second step 6 in., and this section was adopted to Elevation 1020.

In order that a careful analysis might be made of the varying flows over these sections, a tabular form was prepared on which zero was used to indicate that no change has been made in the section; -1 that the tread has been dropped 1 ft., or the riser has been placed 1 ft. farther in than in the normal section; and $+1$, that the tread has been raised 1 ft., or the riser placed 1 ft. farther out. An analysis was also made of the effect of varying heads over the adjusted section, by the use of symbols to represent whether the inner edge of the flow hit heavily or lightly on the steps. This tabulation made it simpler to compare the flows over the varying heads and to analyze their differences.

The foregoing tests were conducted on a 1:20 model. As a verification, a model was constructed of the full height of the dam on a scale of 1:50, and another model, on a scale of 1:8, down to the fourth step. A detailed description of the action over these models will be given subsequently, but it may be noted here, that, although it was not identical, the discrepancy was not excessive.

In extending this section for the full length of the dam, with all the vanes pointing in one direction, water will be piled up against one end. It is thought that, with the higher flows, there will be a tendency for the water to pile up on Step 2, unless some means is provided for admitting air behind the jet at intervals. This might be accomplished by changing the direction of the vanes at intervals, as shown on Fig. 6. Other considerations have led to the decision to make only one change in the direction of the vanes. This change will be made at the west end of the dam opposite the gorge. One hundred and eighty-six different experiments were recorded. This includes flows over one hundred different sections.

Miscellaneous.—As experiments were conducted, tests were made to determine whether a partial vacuum was present in any of the flows. The only place it was found was in the whirl below the blocks or between the vanes. On the 1:8 model, a series of ten readings was taken on each of five different heads, the results of which are given in Table 3.



Comparison of Sections

Height	Elev.	Length	Contr. Section ** Area of Sect.	Contr. Section Area to El.	Revised Section ** Area of Sect.	Revised Section Area to El.	Comparison with Contr. Sect.
0	1130	1324	sq. ft. 0	sq. ft. 0	sq. ft. 0	sq. ft. 0	sq. ft.
6.5	1123.5	1324	——	——	96	96	
10	1120	1323	116	116	——	——	
16	1114	1321	——	——	276	372	
30	1100	1318	731	847	——	——	
31	1099	1318	——	——	578	950	
50	1080	1318	995	1842	964	1914	+ 72
70	1060	960	1255	3097	1275	3189	+ 92
90	1040	660	1534	4631	1554	4743	+ 112
110	1020	165	1932	6563	1874	6617	+ 54
130	1000	160 130	2332	8895	2252	8869	- 26
150	980	125	2751	11646	2671	11540	- 106
156	974	130	938	12584	925	12465	- 119

**Area of section between the elevation given and the elevation next above for the designated section, thus 731 on 5th line = area of section between elev. 1100 & 1120

FIG. 6.

TABLE 3.

Head, in feet.	Negative Pressures, in Pounds per Square Inch.		
	Maximum.	Minimum	Average of ten readings.
3	0.11	0.03	0.07
4	0.15	0.02	0.06
5	0.23	0.11	0.18
6	0.23	0.13	0.19
8.7	0.20	0.07	0.14

It seemed to be the general impression that, on the models without vanes, a negative pressure might be found under the jet from the crest to the first step where the water back of the jet was at a noticeably higher elevation than that in front of it. Numerous tests, however, showed that the entrained air was at atmospheric pressure. It is thought that the following explanation may account for this. When the water first passes over the crest, it follows down the riser. As the head is increased, the water leaves the riser and forms a jet, the section of which is a parabola. As the head is further increased, the curve is extended. Now, if both ends of the jet are sealed, an air-tight compartment is left back of the jet. As this compartment is enlarged by increases of head, there is a tendency for a partial vacuum to form back of the jet, but this is counterbalanced by the atmospheric pressure outside, forcing the water up in the compartment. Thus, as the compartment is enlarged, the water level behind the jet is raised, but the atmospheric pressure is not reduced.

Dissimilarity in Action of Proportionate Flows Over Models of Different Scales

Field Observations.—The dissimilarity, referred to previously, will now be discussed in detail. Fig. 7 shows the relativity of the outer and inner edges of jets for models of Section 16, built on three different scales and tested under heads of 3, 5, and 6 ft. It may be noted that:

1.—A similar divergence of the outer edge of the jet, due to the scale of the model, is found for all three heads.

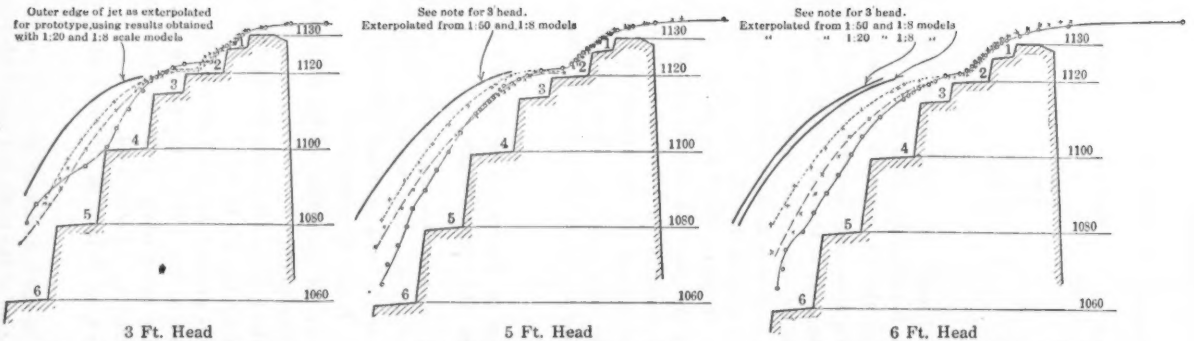
2.—On the crest, the depth of the water is less for the 1:50 model than for the 1:20 model, and greatest for the 1:8 model.

3.—On Step 2, the depth is greatest for the 1:50 model.

4.—Between Steps 3 and 5, the outer edge of the jet, for the 1:20 model, is midway between the outer edges of the jets for the 1:8 and 1:50 models; this is particularly true for the 6-ft. head.

5.—The location of the jet, as defined by its inner edge, varies with the scale of the model.

At first, the discrepancy in the behavior of the 1:50 and 1:20 models was thought to be due to the location of the baffle, but nearly identical results were obtained after moving the baffle, as shown by Fig. 1. Fig. 8 gives definite proof



3 Ft. Head

1:50 scale. The inner edge of jet hits all steps.
1:20 scale. The inner edge of jet hits steps 1 and 2, completely misses step 3; about 5% of flow hits step 4, about 20% hits step 5, entire flow hits step 6.
1:8 scale. The inner edge of jet hits steps 1 and 2, completely misses steps 3 and 4; about 2% of flow hits step 5.

5 Ft. Head

1:50 scale. The inner edge of jet hits steps 1, 2 and 6, completely misses steps 3, 4 and 5.
1:20 scale. The inner edge of jet hits steps 1 and 2, completely misses steps 3 and 4, about 10% of flow hits step 5, about 60% hits step 6.
1:8 scale. The inner edge of jet hits steps 1 and 2, completely misses steps 3, 4 and 5.

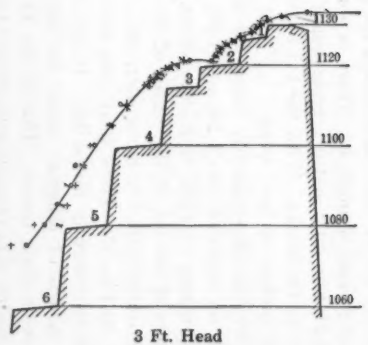
6 Ft. Head

1:50 scale. The inner edge of jet hits all steps except step 3.
1:20 scale. The inner edge of jet hits steps 1 and 2, completely misses steps 3 and 4; about 5% of flow hits step 5, 50% hits step 6, entire flow hits step 7.
1:8 scale. The inner edge of jet hits steps 1 and 2, completely misses steps 3, 4 and 5.

GILBOA DAM MODEL
FLOW OVER EXPERIMENTAL SECTIONS
Scale in Feet
10 0 10 20

Legend
Outer surface of jet, 1:50 scale model
" " " " 1:20 " "
" " " " 1:8 " "

FIG. 7.



3 Ft. Head

Description

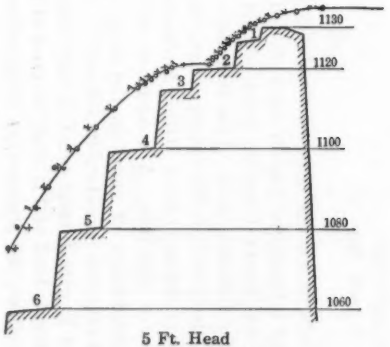
The sections shown are a 1:20 scale model of Section 16.

The difference in identical flows is presumbly due to arrangement and location of baffles built to eliminate surge.

The crosses represent points of actual measurement of Tests A and B; the circles represent the measurements of Test C.

For Tests A and B, the baffle was parallel with, and 5 ft. upstream from, crest of model.

For Test C, the baffle was located at foot of the falls, 80 ft.± upstream from, and approximately normal to, crest of model.



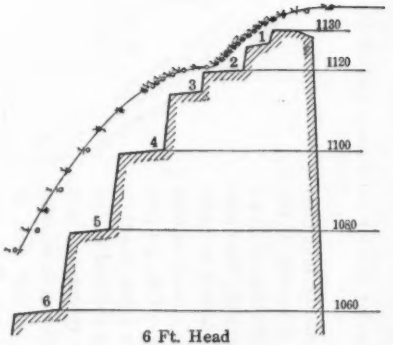
5 Ft. Head

GILBOA DAM MODEL

FLOW OVER EXPERIMENTAL SECTIONS

Scale in Feet

10 0 10 20



6 Ft. Head

Legend

Outer edge of jet, Test A

" " " " " B

" " " " " C

FIG. 8.

of the consistency of the jet under similar conditions and of the accuracy of the measurements.

Fig. 9 shows the measurements taken on jets over the 1:50, 1:20, and 1:8 models of the accepted section, with a proportionate head of 5 ft. On the 1:20 and 1:50 models, the outer edge of the jet shown was determined by holding a straight-edge level in such a position that the outer part of the outside face of the jet just touched the straight edge. This point was then projected on the board forming the end of the model. The circles in Fig. 9 represent these points. On the 1:8 model, the points were located in a similar manner, except that it was found more practical to locate the point by eye than by the use of a straight-edge. A part of the jet hit the tread of each step on all models.

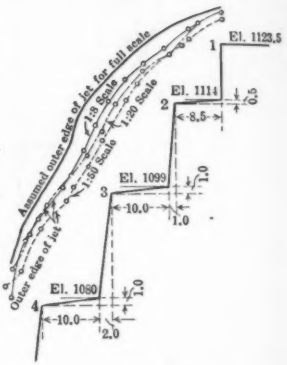


FIG. 9.

It was found by measurements in the field that the outer edge of the jet of the 1:20 model is about midway between the jets on the 1:50 and 1:8 models. From this, by interpolation, it was assumed that the outer edge of the jet may be approximately located for the full-scale dam. This assumption is checked theoretically by the fact that the force of the falling water, which projects the spray, increases as the cube of the scale ratio. Other forces, however, may materially change the location of this line. A comparison of Fig. 9 with Fig. 7 will show, that the introduction of vanes in the upper part of the section, by diminishing the velocity, has considerably reduced the dissimilarity due to scale. Figs. 10, 11, and 12 give further proof of this dissimilarity, by showing the effect of a proportionate head of 3 ft. over the crest of the accepted section on models of all three scales, the photographs being enlarged to show the models the same size.

The water passes over the vanes of the 1:50 model (Fig. 12) with no perceptible break-up. The surface of the jet is so smooth that the grain of the wood in the vanes can be readily seen through it. There is some break-up of the jet with the 1:20 model (Fig. 10), and a certain quantity of spray is cast out, as the water falls over the blocks. With the 1:8 model (Fig. 11), a large proportion of the jet is broken up into spray.

In order to test further for this dissimilarity, three flat-crested weirs, with a free overfall, were built on scales of 1:8, 1:20, and 1:50, with a proportionate length of 30 ft. Sections of these weirs are shown on Fig. 1, and the outer and inner surfaces of the jet, at the center of the weir, under heads of 3, 5, and 6 ft. as measured with the rectometer, and plotted on a proportionate scale, are shown on Fig. 13. These three jets, although following the same general line, are consistently dissimilar. It will be noted in comparing the jets over the models of different scales for any head, that there is a consistent divergence in their traces. This is due to the proportionately different effects of viscosity, skin friction, and surface tension, owing



FIG. 10.—GILBOA DAM, SCALE 1:20, PROPORTIONATE HEAD, 3 FT.



FIG. 11.—GILBOA DAM, SCALE 1:8, PROPORTIONATE HEAD, 3 FT.



FIG. 12.—GILBOA DAM, SCALE 1:50, PROPORTIONATE HEAD, 3 FT.







to the scale of the model. It will also be seen that this divergence is greatest for the 3-ft. head and least for the 6-ft. head, because the proportionately different effects of viscosity, etc., are greater at low heads.

Theory of Models.—In the Century Dictionary, the word, model, is defined as:

"A detailed pattern of a thing to be made; a representation, generally in miniature of the parts, proportions, and other details to be copied in a complete production.

"A mechanical imitation or copy of an object generally on a miniature scale, designed to show its formation; hence, an exact reproduction; a facsimile."

In the following discussion, a model will be considered to be in exact geometrical similarity to its prototype, with all linear dimensions in the same scale ratio.

The late William M. Torrance, M. Am. Soc. C. E.* took up the design of static structures from models, and pointed out that a model water tank on a scale of 1:30 was proportionately thirty times stronger than its full-scale prototype. Fig. 14 shows similar computations for a water tower, a beam, and a gravity dam, the principle being the same in all cases, namely, that the unit strength varies inversely with the scale ratio. In dynamic models, the principal causes of dissimilarity in action are: Force, velocity, viscosity, and atmospheric pressure.

B. F. Groat, M. Am. Soc. C. E.,† presents the following formula for extraneous force ratio: For the proportionality of force ratio between the model and its prototype, when the tests are conducted on the same place on the earth's surface, the ratio of the densities of the two substances equals the ratio of the cube of the linear dimensions. If the same substance is used for both model and prototype, the extraneous forces are proportional to the cube of the scale ratio.

This may be illustrated by the following numerical example: Visualize a 4-ft., solid, wooden cube, weighing 56 lb. per cu. ft., and a model of the same material on a scale of 1:2. The large block will weigh 3 584 lb. and the small block, 448 lb. If the larger block falls 20 ft., its velocity will be 35.87 ft. per sec., and the time required will be 1.12 sec. The force, $\frac{\text{velocity} \times \text{mass}}{\text{time}}$‡ = 115 820 ± ft-lb. per sec. Using the same linear scale ratio, the smaller block will fall 10 ft., and its velocity will be 25.36 ft. per sec., the time, 0.789 sec., and the force, 14 400 ± ft-lb. per sec. Thus, the ratio for the model and its prototype will be 1:8, or as the cube of the scale ratio.

If the smaller block is made of cast iron weighing 448 lb. per cu. ft., and the scale ratio remains the same, the weights will vary as 1:8, or as the cube of the scale ratio, and both cubes will weigh the same. If the smaller block falls 10 ft., its force will be the same as the larger block falling 20

* "Use of Models in Engineering Design", *Engineering News*, December 18, 1913.

† *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), pp. 1181 and 1190.

‡ "Civil Engineers' Pocket Book", John C. Trautwine, 1904, p. 337.

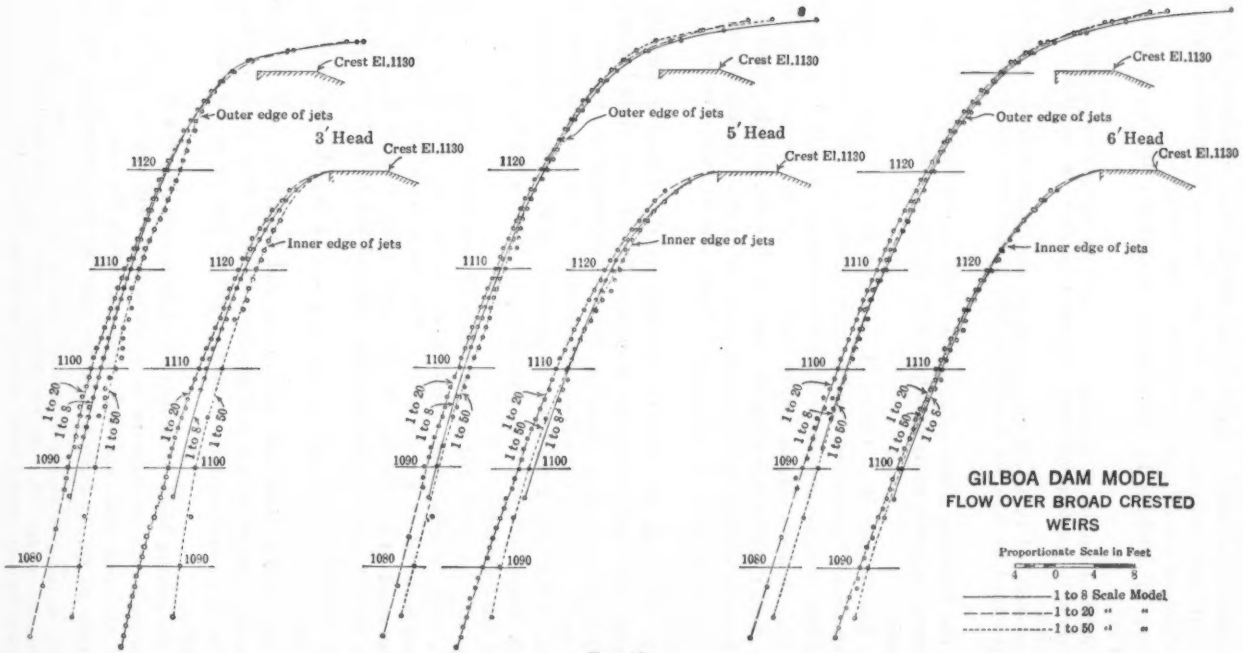
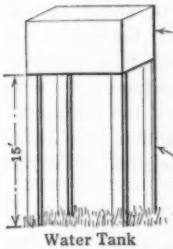

Crest El.1130
Outer edge of jets
3′ Head
Crest El.1130
Inner edge of jets
1120
1110
1100
1090
1080
1 to 20
1 to 8
1 to 50
5′ Head
6′ Head
GILBOA DAM MODEL
FLOW OVER BROAD CRESTED WEIRS
Proportionate Scale in Feet
4 0 4 8
1 to 8 Scale Model
1 to 20 " "
1 to 50 " "

FIG. 13.

ft. Thus, if materials of different densities are used, to obtain an equal force, the scale ratio for the model and its prototype will be the cube root of the densities.



Water Tank

Water Tank

(1) Compute stress in lb. per sq. in. on lower end of post where it enters the ground

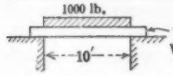
Weight of water = 10 x 10 x 5 x 62.5 =	31 250	lb.
" " tank = 0.5[(5 x 4 x 10.5) + 11^2] x 50 =	8 275	lb.
" " posts = 15 x 1 x 1 x 4 x 50	3 000	lb.
Total Weight	42 525	lb.
Weight per post	10 631.2	lb.
Weight per sq. in.	73.8	lb.

(2) Compute same data for a model built on a scale of 1:10

Weight of water 1 x 1 x 0.5 x 62.5	31.25	lb.
" " tank = 0.05[(0.5 x 4 x 1.05) + 1.1^2] x 50 =	8.275	lb.
" " posts = 1.5 x 0.1 x 0.1 x 4 x 50 =	3.00	lb.
Total Weight	42.525	lb.
Weight per post	10.631	lb.
Weight per sq. in.	7.38	lb.

Results

Weight varies as the cube of the scale ratio
Area " " " square " " " "
Strength " inversly as " " "
i.e. the unit strength of the model is ten times that of its prototype.



Simple Beam

Let W = weight of load and beam = 1344 lb.
L = length of span = 120 in,
b = width of beam = 10 in. d = depth of beam = 10 in,
s = stress in lb. per sq. in.

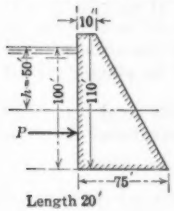
$\frac{WL}{8} = \frac{sbd^2}{6}$ s = 121 lb.

For model scale 1 to 10 W = 1.34 lb. L = 12 in. b = 1 in.

$\frac{WL}{8} = \frac{sbd^2}{6}$ d = 1 in. s = 12.1 lb.

Results

Weight varies as the cube of the scale ratio
Area " " " square " " " "
Strength " inversly as " " "



Weight of Masonry 125 lb. per cu. ft.

Dam (Stability)

Full Scale	Model
$P = ahw$ = 6 250 000 lb.	$P = ahw$ = 6 250 lb.
Weight of dam = 11 687 500 lb.	Weight of dam = 11 687.5 lb.

Therefore stability against overturning is the same in the model and prototype

Dam (Pressure on Base)

$$\text{Unit Press} = \frac{P + \text{Weight of dam}}{\text{Area of base}} = \text{11 958 lb. for full scale and}$$

1 195.8 lb. for 1:10 model, thus the unit pressure is inversly proportional to the scale ratio

FIG. 14.

Mr. Groat* gives a theoretical discussion on the proportionality of the velocity in a model and its prototype. Briefly, it is as follows:

By the Chezy formula for the flow in channels or rivers, $v = c\sqrt{rs}$; let this equation stand for the flow in the model, and let $V = C\sqrt{RS}$ be the equation for flow in the full-size prototype. Then, $\frac{v}{V} = \frac{c\sqrt{rs}}{C\sqrt{RS}}$ will show the relation between the two flows. If both the model and its surfaces are

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1173.

geometrically similar to the prototype, then $\frac{c}{C} = 1$ and $\frac{s}{S} = 1$. Squaring and eliminating, $\frac{v^2}{V^2} = \frac{r}{R}$; therefore, $\frac{v}{V} = \frac{\sqrt{r}}{\sqrt{R}}$, showing that the "velocities should be proportional to the square roots of the hydraulic radii, and, therefore, to the square roots of any homologous linear dimension", or to the square root of the scale ratio. It can be shown that this will apply equally well to a weir. Thus, if a model of a weir or channel is geometrically and mechanically similar to its prototype, the velocity will vary as the square root of the scale ratio. Generally speaking, it will be found difficult in the construction of practical weirs to make the ratio of the two coefficients of roughness equal unity and, consequently, $\frac{c}{C}$ will not exactly equal unity.

The velocity of the wind will vary in the same manner as other velocities, that is, as the square root of the scale ratio. A 5-mile breeze on the model, on a scale of 1:50, will be equivalent to a 35-mile wind on the full-scale prototype.

Mr. Groat* states: "This shows that cases of pure stream-line flow cannot be reproduced in models unless the sizes of the models are properly determined by Equation (19)". This equation is, $L^{\frac{3}{2}} = N(G)^{-\frac{1}{2}}$, in which L equals the scale ratio, N the ratio of the kinematical coefficients of viscosity corresponding to the coefficients of viscosity of different fluids used in the model and the prototype, and G equals the gravity ratio.

"As examples of the uses of Equation (19), when $G = 1$, it may be shown: (*a*) that a model air propeller in mercury at 20° cent. should be one-twenty-fifth full size for the prototype in air at 15° cent.; (*b*) that the scale ratio of a model water-wheel tested in mercury at 20° cent., should be about 0.22 for the prototype in water at 15° cent. * * *"

"* * *. With $G = 1$, Equation (19) shows that the model requires a different fluid from that for the prototype when both observations are made at the same place."

Thus, to give the same results as its prototype, an overfall dam should have its model tested with some liquid other than water.

Under ordinary conditions of testing, the atmospheric pressure is the same for both the model and its prototype; thus, in the prototype the atmosphere will support a column of water 33.9 ft. high. As the atmospheric pressure remains practically a constant, it will support the same height of water in a model regardless of the scale. Thus, in a model on a scale of 1:50, the atmosphere would apparently support a column of water 1 695 ft. high. Unless some means are taken to reduce this pressure, proportionately to the scale of the model, in hydraulic flows in which there is a partial vacuum, or negative pressure, the action of the model will differ from that of its prototype.

Application of the Theory of Models to the Models of the Gilboa Dam.—Probably the principal cause for the dissimilarity below the crest in models

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), pp. 1186–1187.

of various scales, is the variation in force due to the scale of the model. It has already been shown that this varies as the cube of the scale ratio. Thus, the force exerted by any part of the jet falling on the 1:8 scale model is 2.5^3, or 15.6 times greater than the force exerted by the homologous part of the jet falling on the 1:20 model, or $6.25^3 = 244.1$ times greater than on the 1:50 model. This force is largely expended in breaking up the jet and in causing the particles of the jet to be projected farther.

It is believed that this explains the difference in the action over the vanes, as shown in Figs. 10, 11, and 12. A study of these photographs will show that, in the 1:50 model (Fig. 12), the surface of the jet passing over the vane and first step is nearly smooth. The impact is so slight, as the water passes over the edge of the vane and falls on the tread of the first step, that there is scarcely any noticeable break-up.

On the 1:20 scale (Fig. 10), where the fall of the water exerts a force 15.6 times greater than on the 1:50 model, a considerable break-up is noticeable where the water falls over the vanes and crest and impinges on the first step.

On the 1:8 scale (Fig. 11), the force exerted by the falling water is 15.6 times greater than in the 1:20 model, and 244.1 times greater than in the 1:50 model. In this case, there is a marked break-up in the jet as it flows over the first step. In no place is the smoothness so noticeable as in the 1:50 model, or even in the 1:20 model.

Some idea of the break-up which will occur in the full-scale prototype, may be obtained by first comparing the 1:50 and 1:8 models, in which the force ratio is 1:244.1, and then attempting to visualize the full-scale model in which the force will be 512 times the 1:8, or 125 000 times the 1:50, model.

The dissimilarity in models below Step 1 is probably best shown on Fig. 9. On this diagram, the outer edge of the jet has been plotted for a proportionate head of 5 ft. It will be noted that, generally speaking, the difference in force has had the effect of placing the outer edge of the jet a little farther away from the weir as the scale increases. The experiments seem to show, however, that, in this model, the center of the jet remains in approximately the same location for all heads and that the break-up simply increased the area of the jet. In this connection, it will be noted that the area of the jet flowing over Step 1 is larger in the 1:8 scale than in the 1:20 scale. (See Fig. 9.)

It must be noted, however, that below Step 4 this is only conjecture, as the 1:8 model did not extend below that point. It is thought that the outer edge of the jet on the full-scale model will be proportionately farther out, or, in other words, it may be exterpolated from the location of the jets on the three models. This is assumed by noting that the general location of the outer edge of the jet of the 1:20 model is midway between the 1:8 and 1:50 models.

As a further check on the theory that the dissimilarity in models of stepped overfall dams is caused partly by variation in force exercised by the falling water, due to the scale of the model, reference is made to the studies of the flow over Section 16.

Between Steps 3 and 5, the location of the outer edge of the jet varies with the scale of the model, its distance from the dam being greatest for the 1:8 scale. Probably the principal cause for this is that the energy of the jet, as it impinges on Step 2, is greatest for the 1:8 model, thus causing the jet below that point to be projected farther than in the other models.

From measurements on flows over the flat-crested weirs, taken in the field, it was possible to determine with considerable accuracy the area of the jet on the down-stream edge of the crest, and with this area and the computed quantity, the velocity was calculated. Table 4 shows the computed velocities, in comparison with the theoretically proportionate velocities. The theoretically proportionate velocities were determined by assuming the velocity for one scale as standard, and computing the other by multiplying or dividing by the square root of the scale ratio.

TABLE 4.

Model.	Computed velocity, in feet per second, assumed as actual velocity.	Assuming 1:8 model as standard.	Assuming 1:20 model as standard.	Assuming 1:50 model as standard.
1:8	4.99	.....	5.02	5.11
1:20	3.18	3.16	.....	3.23
1:50	2.045	2.00	2 01	.....

It will be noted, that the proportionate velocities shown in Table 4, do not differ greatly from the velocity assumed as correct, but they do differ sufficiently to change the curve of the jet so that it will not check the curve measured in the field.

As already stated, therefore, the writers conclude that flows over models of broad-crested weirs, although giving results that approximate closely those over the prototype, do not show an identical similarity, owing to the fact that the coefficient of flow over a broad-crested weir is not always proportional to the ratio of head and width of crest, due largely to the disproportionate effect of skin friction and viscosity.

By computation, it can be shown that by changing the geometrical similarity to a slight degree, that is, by using a corrected head, flows may be obtained which are identically similar for all models. Thus, if instead of a 6-ft. proportionate head (0.75-ft. head (actual scale), on the 1:8 model), a head of 0.69 ft., actual scale, is substituted, a quantity is obtained that corresponds with the quantity computed for the full-scale model. For the 1:20 scale, the head will be 0.255 ft. instead of 0.30 ft. (both actual scales), and for the 1:50 scale; 0.10 ft. instead of 0.12 ft. These results have never been checked in the field, but are thought to be fairly accurate as the retardation of velocity, due to a broad crest, increases with the scale of the model.

Owing largely to the effect of skin friction and viscosity, the jets over model flat-crested weirs do not exactly check each other. A description of these weirs and the tests on them has already been given. It remains to prove theoretically why these jets do not follow the same line when plotted on a proportionate scale (Fig. 13). If these jets followed the same line, the

velocities would be in proportion to the square root of the scale ratio, and the coefficient, $\frac{c}{C}$, would equal unity. (See page 297.)

The following paragraph is quoted from a paper by Richard R. Lyman, Assoc. M. Am. Soc. C. E., entitled, "Measurement of the Flow of Streams by Approved Forms of Weirs with New Formulas and Diagrams,"* in which a graph is shown giving heads and corresponding discharges over broad-crested weirs:

"It should be noted that, for comparatively low heads, all these broad-crested weirs give practically the same result for the same head. At a particular head or point, however, for each width of crest, the quantity of discharge begins suddenly to increase much faster, in proportion to the head, than it has done before, and the line representing its discharge breaks away from *AA* and extends across the diagram [Plate XXIV] in a slanting direction to the line, *BB*, which represents the discharge over weirs with sharp crests."

A study of this statement and the graph will lead one to believe that the models of broad-crested weirs, on different scales, will not check. If they were identically similar, the lines representing width of crest would be straight and not broken. If the action of a model is precisely similar to its prototype, then, in the formula for discharge over broad-crested weirs,

$$Q = C\ 3.33\ L\ (H)^{\frac{3}{2}},$$

C, which depends on width and head, should be a constant for the model and its prototype, regardless of the scale of the model, as in the model the ratio of width and head would be the same as in the prototype. A table† compiled by Gardner S. Williams, M. Am. Soc. C. E., from the Cornell experiments on flow of water over weirs, will prove that this is not true. As an example: With the head of 2 ft., and a width of crest of 1.65 ft., the coefficient is 0.925. Multiplying both head and width of crest by 2, the coefficient is 0.807.

As a further check, the flow over a weir 30 ft. long, 6 ft. wide on the crest, with a head of 6 ft., with end contractions, was computed, using the formula,

$$Q = 2.64\ (L - 0.2\ H)\ H^{\frac{3}{2}};‡$$

for the models, the following formula was used,

$$Q = 3.33\ (L - 0.2\ H)\ H^{\frac{3}{2}},$$

with the exception that for the 1:8 model a correction was applied for broad crest. These computed flows were then compared with proportionate flows obtained by the formula,

$$\frac{Q}{q} = \frac{A\sqrt{2\ G\ H}}{a\sqrt{2\ g\ h}}.$$

The results of these computations are shown in Table 5.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1199, and Plate XXIV, p. 1205
† "Waterworks Handbook", by Messrs. Flinn, Weston, and Bogert, Table 196, p. 601.
‡ Merriman's "Hydraulics", p. 161.

TABLE 5.

Model.	Computed flow, in second-feet.	Comparison with full scale, second-feet.	Comparison with 1:8 scale, second-feet.	Comparison with 1:20 scale, second-feet.
Full scale.....	1117.7	.....	.....	.....
1:8.....	7.018	6.16	.....	.....
1:20.....	0.7864	0.625	0.7107	.....
1:50.....	0.081	6.059	0.064	0.073

This proves that, in models of broad-crested weirs, as $\frac{c}{C}$ does not equal unity, mechanical and geometrical similarity cannot be obtained simultaneously, owing, in all probability, to the proportionately different effect of skin friction and viscosity. In determining the proper quantity for this tabulation, it seems best to use that quantity, the computed jet of which plotted nearest to the actual jet as measured in the field.

For each flow, the quantities were computed by each of the following formulas: Bazin, Fteley and Stearns, Francis, and U. S. Geological Survey. Using the velocity derived from these formulas, paths of the jet were plotted by applying the jet formula,

$$x = vt,\ y = \frac{1}{2}\ gt^2,$$

and compared with the curves that had been determined in the field by locating the inner edge of the jet. The accepted quantity for each flow was that one the jet of which was nearest to the jet located in the field. Similar computation sheets were made for the 1:20 and 1:50 models.

For comparison, the theoretical parabola, derived from the formula:

$$x = \frac{2}{3}\sqrt{2\,g\,h}\ t^*,\ y = \frac{9}{16}\frac{x^2}{h},$$

was plotted. This line was considerably under the inner edge of the jet as measured, and, therefore, does not represent as great a velocity.

The main causes, therefore, for the dissimilarity in action over these models are: Difference in force; difference in coefficients of flow over a broad-crested weir, due to skin friction and viscosity; and difference due to atmospheric pressure, when parts of the flow are not open to free circulation of air.

Effect of Dissimilarity of Models on Experiments with Flow in Channel.— In the final experiments with the flow in the channel, Section 16 of the dam, on the 1:50 scale, was used. No experiments were made with models of any other scale, nor have experiments been made using the accepted section (Section 100) of the dam. It is thought that the revised section will tend to bring the water to the floor of the channel with a lower velocity than Section 16. However, the force exerted by the falling sheet of water will be 125 000 times greater than that in a model on the scale of 1:50. With the present limited knowledge, it would be inadvisable to predict just what the action in the channel will be.

* Encyclopædia Britannica, Vol. 14, p. 51.

General Conclusions

From observations, measurements made in the field, careful study of photographs, and a study of the theory of models, the writers conclude:

1.—That the flow over a large scale dam will be similar to a certain degree, but not identical, with the flow over its models.

2.—That, as Mr. Groat has pointed out, it may be possible to build a diminutive structure, not necessarily a model in the true sense of the word, which "must reproduce, even in detail, the performances of their prototypes", but that, in many cases, the practical difficulties encountered may make the construction and test of these models impractical. For instance, in order to obtain a proportionate test for the force of the falling water on the steps of the Gilboa Dam, it would be necessary to build a model on a scale of 1:2.38 and test with mercury at 60° Fahr.

3.—That, with a full recognition of the fact that there is dissimilarity, the model, if built on a large enough scale, still remains the most efficient method of determining the most satisfactory section for an overfall dam.

DISCUSSION

GEORGE G. HONNESS,* M. AM. SOC. C. E. (by letter).—The authors briefly state that 186 different experiments were recorded, which include flows over 100 different sections. These experiments were started in the early summer of 1919, and were continued until weather conditions in the early winter caused the work to be stopped before conclusions could be reached. A complete reconstruction of the control works and parts of the model was required in the spring of 1920 before experiments could be resumed, and then the experiments were continued until the early fall of 1920. This brief statement is not, therefore, a fair measure of the faithful, painstaking, and ingenious work done by the authors, for which they are to be commended.

As stated in the paper, the original purpose of the construction of the model was to determine the sufficiency of the spillway channel, as it was thought by some that it was more than ample, and that a considerable saving would result by a reduction in size.

The behavior of the model on a scale of 1 : 50, with a maximum flow equivalent to a 6-ft. head over the full-scale dam, indicated that a reduction of both width and depth was permissible. Experiments were then made with a width of channel reduced by 50 ft., with a head equivalent to 6 ft. on the full-scale dam, and the results are shown in Fig. 4. This shows that the reduced channel, with assumed flood flows, would probably be ample.

The flood flow of Schoharie Creek, at Gilboa, in 1869, was the highest in the memory of the then oldest inhabitant. It seems fair, therefore, to assume that the flood of 1869 was the highest in more than 100 years. A fair determination of the quantity of that flood at Gilboa was 44 100 cu. ft. per sec., or 140 cu. ft. per sec. per sq. mile, from 314 sq. miles. As stated, the channel of reduced size was shown to be ample for a flood of 51 381 cu. ft. per sec., or 163 cu. ft. per sec. per sq. mile. Notwithstanding, the channel is to be constructed as originally planned, for the reasons stated by the authors.

A more important fact than the capacity of the channel was determined by the initial experiments. With a section as originally designed, having steps of unusual height and width of tread for an overfall section, the flow passed over the crest, down the first riser, over the tread, and then the sheet of water missed the next three or four steps, falling as much as 80 ft. Hence, the necessity of continued experiments until an overfall section could be selected, which would lead to the falling sheet impinging on each successive step and thus dissipate the energy of the falling water. This was finally accomplished by placing deflecting vanes on the tread of the upper step and modifying the height and width of the other steps.

The writer concurs in the authors' conclusion, that notwithstanding the fact that the behavior of models will not be exactly similar to their prototypes, the model remains the best means of determining the design of many hydraulic structures. This conclusion is based on experience with models used to

* Dept. Engr., Board of Water Supply, New York City, Grand Gorge, N. Y.

determine the probable behavior of full-scale structures in the following instances.

Boonton Dam, Boonton, N. J.—The overfall section of this dam was located on the northerly side of the valley, so that the maximum height was not in excess of 70 ft. This required the spillway channel to make an acute angle with the axis of the dam, in order to lead the waste water to the old stream bed. The overflow from the northerly one-third of the waste weir is caught on a steep, sloped channel the maximum grade of which is parallel to the axis of the dam. The water, flowing over the steep slope, has its velocity accelerated, and this deflects the water passing through the southerly two-thirds so that it follows the axis of the spillway channel. A model was constructed to scale, and the plans were perfected after studying the action of the water passing over the model. As constructed, the channel has served its purpose, and the result is substantially the same as that obtained with the model.

During construction, the channel was advanced so that it could be used during the winter season to care for water passing over the overflow section which, as the end of the working season approached, was in such a condition that a concentrated flow over a small section of its length would pass into the channel. A model was constructed to comply with conditions as they existed, and flows over it showed that the side of the channel would be overtopped and serious damage likely to result. A modification which increased the length of the temporary overflow section several times was tried, and the contractor was required to advance the work sufficiently to bring about the conditions desired. Overflows during the winter and the following spring did no damage.

Cross River Dam, Katonah, N. Y.—Slight modifications of the waste weir and spillway channel of this dam were determined from experiments with a model section.

Ashokan Reservoir Waste Weir, Ashokan, N. Y.—The spillway channel changes the direction of flow so that it makes a right angle with the line of the waste weir. The north retaining wall contained a re-entrant angle which deflected a part of the flow, so that it impeded the flow of that part of the water moving parallel to the waste weir section. A discharge due to a head of slightly more than 1 ft. on the crest of the weir, made it apparent that modifications were necessary to meet conditions of larger flows which were to be anticipated. A model on a scale of 1 : 50 was constructed, and from its operation it was decided to remove the re-entrant angle and construct a guide-wall in the center of the channel at the right-angle bend in its alignment. No overflow has occurred since this work has been done, but it is predicted that the channel will perform its purpose more efficiently than as originally designed.

B. F. Groat,* M. Am. Soc. C. E. (by letter).—In reading the authors' instructive paper, the writer has noted that the overfall did not correspond exactly with their expectations. The question may well be asked—should it? Even if model theory was developed sufficiently to be closely applicable, no one need expect such close agreement that errors will not become apparent. This

* Cons. Engr., Philadelphia, Pa.

is true of all theories as regards exact application. In particular, the writer is of the opinion that an overfall is very sensitive to small errors, such as might occur in the measurement of the head on the crest of a model weir.

One element of correct hydraulic model construction which is difficult to create when the prototype does not exist, is a correct introduction of water to the model with corresponding correct configuration of discharge. If these two conditions are not established, an exact reproduction of performance need not be expected. In the case of the writer's model of a reach of the St. Lawrence River, the prototype actually existed, yet, owing to the roughness of the field methods used, errors of some magnitude were observed, not only in the flow within the model, but also in the attempted reproductions of entrance and exit conditions. Had these conditions been minutely adjusted, there is little doubt that a more precise homology would have been presented.

To be even partly true, the statement that the strength of model structures "varies inversely as the scale ratio", must be qualified by the word, "geometrical". When the extended meaning of the word, "model", is used, referring now to combined geometrical and mechanical similarity, no such restricted statement can be applied. In short, the moment the conditions of mechanical similarity are superposed on the conditions of geometrical similarity, the theorem of the statement becomes a "cull". The word, "model", must have an extended meaning or progress in research is at an end as regards models.

In the application of rigorous theory, the construction and testing of a model becomes a large problem. Is a valuable method of solution, however, to be dropped because it is large and troublesome? The chances will be that it is not as large and troublesome as the problem to be solved. The expenditure of time and trouble will be justified by the exactness of the results obtained. To make a model structure, for example, one may increase gravity effects by additional density, by loading, by the use of materials of correspondingly varied elastic properties and strength, by centrifugal forces produced by the rotation of the model as a whole, by measuring internal stresses in a model member instead of testing to destruction, etc. Some genius will arise to synthesize materials and fluids capable of being handled with facility in the construction and testing of models. A special study of this kind should be made.

ROBERT FLETCHER,* M. AM. SOC. C. E. (by letter).—This paper supplies in great detail the actual results looked for by B. F. Groat, M. Am. Soc. C. E., in his paper on "Ice Diversion, Hydraulic Models and Hydraulic Similarity",† which the writer discussed.‡

All readers familiar with the principles involved must be impressed by the great value of these remarkable demonstrations of what may be learned by the intelligent use of models. In the discussion referred to, the writer objected to what seemed to be too positive and all-inclusive claims for the validity of model studies, and probably over-emphasized the objections without expressing due recognition of the value of such studies when results are rightly interpreted.

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† *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1168.
‡ *Loc. cit.*, pp. 1156–1162.

The thanks of the Profession are due to Mr. Groat for his very complete exposition of the theory of hydraulic models. It removes some points of haziness, which the writer confesses existed in his own mind. The interesting experiments with models of the Gilboa Dam show how much may be learned practically by the cut-and-try method.

The writer thinks, however, that the main point and intent of his criticism is justified, in all ordinary cases where experimenters have only limited means at their command and are likely to place too much confidence in results which are not conclusive. The final conclusions of the authors from the application of the formulas to the latest experiments are:

First.—A similarity to a certain degree, but not identity, in flow over the actual dam and the model.

Second.—It may be possible to build a model not necessarily a model in the true sense of the word, which "must reproduce, even in detail, the performances of their prototypes;" but, for the Gilboa Dam, the scale would have to be as 1:2.38, and the liquid must be mercury at 60° Fahr.

Third.—With due recognition of the points of dissimilarity, the model, on a sufficient scale, is the most efficient means of preliminary study of such a case as that of the Gilboa Dam.

No one would presume to dissent from the last conclusion in view of the practical demonstrations made by Mr. Groat and the authors. The second conclusion, however, is an admission of practical impossibility.

If the reason for the difficulties in adjusting the theoretical demonstration to practical conditions is considered, it is noted that it involves inclusion of mass ratio, force ratio, centrifugal reactions (in hydraulics), proportionality of heads and depths, skin frictions, wind friction, movement and transportation of detritus in river hydraulics, viscosity, duration of time observations, and factors of safety. Although all these influences are given mathematical expression in the formulas, there are certain coefficients of uncertain value involved, which forbid confidence when an attempt is made to predict what may happen when an extraordinary flood is pouring over a dam, or provision is to be made for a jam of ice or logs, or excessive wave action is to be resisted by a sea-wall, or the impact due to reaction from back-water is to be resisted at the toe of a dam, or in other cases where the forces of Nature are likely to get beyond control. As Mark Twain said, "we are harassed by doubts".

The elaborate German experiments, quoted from the work on river hydraulics* by J. L. Van Ornum, M. Am. Soc. C. E., in the writer's discussion previously mentioned, resulted in great uncertainties and lack of very definite conclusions, although useful in some respects. The artifice of using bags of shot and pieces of slate for groins and bank protection in the models surely did not realize conformity in mass to materials available for actual constructions on the river.

To be consistent, one must believe that it would be possible to make a model of Niagara Falls, which would reproduce on a reduced scale proportional effects to the erosion and destruction wrought by the mighty cataract, or that

* "The Regulation of Rivers," pp. 163–172.

officers of the U. S. Ordnance Department could make a model of a 16-in. gun and invent an explosive of detonating effect proportional in the scale ratio to the effect of the actual high explosive used in warfare, and thus predict the safety of the large gun.

In discussing the application of the formulas expressing "the inexorable laws of mechanics" to the model of the Meigs elevated railway, Mr. Groat shows, perhaps conclusively, that, after a certain number of "ifs" had been duly satisfied, "the factors of safety in model and prototype would have been equal", and then adds: "It is not doubted that it would be something of an undertaking to construct and test such a model, but the fact which we have proved is that it can be done." Again, the verdict is, not really practicable because of inherent limitations.

Acknowledging the great value of all the experiments described, which have specific ends in view, the writer would disclaim any intention, both now and heretofore, to under-estimate the value of the judicious use of models. The caution is against the misuse of them, especially by inexperienced experimenters.

I. GUTMANN,* ASSOC. M. AM. SOC. C. E. (by letter).—In 1919, the writer was instructed by G. A. M. Elliott, M. Am. Soc. C. E., Chief Engineer of the Spring Valley Water Company, of San Francisco, Calif., to prepare a tentative design for a spillway for the Calaveras Dam. The spillway was to take care of a drop of about 200 ft.

An extensive preliminary study, covering more than seventy reservoir spillways in all parts of the world, was made in connection with this design. Special attention was given to stepped spillways or cascades, that is, the type used on the Gilboa Dam.

Stepped spillways and wasteways have been used in European† and American‡ practice for many years, yet the principles for their design have not been formulated, and the writer had to state that there was no reliable formula for the flow of water over cascades. The absence of water-cushions and the consequent progressive increase in velocity of approach preclude a satisfactory analysis. At low heads, each step may be treated as a drop, with no raised crest, and the formula, $Q = 4.75\ L\ H^{\frac{3}{2}}$, may be used. German engineers prefer the Kutter formula. At high heads, with the water being projected farther horizontally and not all the nappe striking the next step, the problem will become more complex and the formulas preposterous.

The writer has been especially interested in this valuable paper which presents the results of the authors' experiments to determine the conditions of flow over the stepped overfall dam at Gilboa, N. Y., by means of a set of models.

Experiments with models could not be expected to furnish very accurate results in this particular case which is essentially dynamic and involves so

* Care, Elec. Bond & Share Co., New York City.

† Crosswood and Roseberry Reservoirs in Scotland: also, at Sengbach, Urft, Bystricka, Krauserbauden, Möhne, etc., in Germany and former Austria-Hungary; and at Rochebut and others in France.

‡ Titicus, New Croton, and others of New York Water Supply; also, the Lahontan Reservoir of the Truckee-Carson Irrigation Project.

complex a phenomenon as the successive impingements of a sheet of water of varying head on masonry steps of varying width and height. Considering the 1 : 8 model (Fig. 7), as a prototype, and the 1 : 20 and 1 : 50 models as its 1 : 2½ and 1 : 6¼ reductions, one will conclude that it will require unusual intuition to exterpolate the curve of flow over the prototype from observations on the models, especially in the case of ordinary heads, namely, less than 4 ft. or more.

More consistent and, therefore, more serviceable results were obtained with the models of the "approved Section 100" (Fig. 9). It may be said, perhaps, that Section 100 was approved because the models of it gave more consistent results, these results being due probably to the greater uniformity of its profile. The rises of the steps in Section 100 are consistently greater than their treads, whereas, in the experimental sections, the rises are smaller than their treads, in the uppermost three steps and greater in the steps below the third, therefore, in Step 2, the ratio of rise to tread is 1 : 1½, and in Steps 5 and 6, the ratio is 2 : 1.

Most of the "discrepancy" or "dissimilarity" with the experimental models probably originated on Step 2, the tread of which was long enough to deflect and to project the falling nappe in a horizontal direction, whereas the profile of the model, near that point, takes a vertical deflection downward with reference to its original trend. The flow curve diverges from the profile, and the dissimilarity between the models, which becomes prominent below this point, is probably due chiefly to the mass effect of their respective volumes of water which had impinged on and were deflected by the tread of Step 2.

In the "approved Section 100", the horizontal deflection of the nappe taking place on Step 2 of the experimental sections, was suppressed by shortening the tread of Step 2 and by a general steepening of the upper part of the profile of the dam. This made the profile more uniform in respect to rise and tread proportions, the structure becoming geometrically simpler and the results more consistent.

Better results were obtained by the authors in their tests of models of broad-crested weirs, the obvious reason being the greater simplicity of the structures and the absence of the complex impingement factor. In connection with these tests, the authors made the interesting observation that kinetic dissimilarity may be eliminated "by changing the geometrical similarity to a slight degree, that is, by using a corrected head, flows may be obtained which are identically similar for all models", irrespective of their scale. The corrections computed by them for the three models used seem to be consistent when properly plotted against the scales of the three models. Curves based on such observations might supply a very useful link for the correlation of observations on models with what may be expected to occur on full-scale structures.

In general, the study tends to show that the method of models, if rationally and cautiously used, can be reasonably depended on even in quantitative studies of proposed broad-crested weirs and other hydraulic structures not unduly complicated.

The authors deserve great credit for their thorough investigation and their valuable contribution to hydraulic literature.

ROBERT F. EWALD,* ASSOC. M. AM. SOC. C. E. (by letter).—There is much to commend and little to criticize with respect to the methods used in and the conclusions obtained from the admirable series of experiments made on models of the stepped profile type of dam. The general type of dam to be used apparently had been decided on before the experiments were undertaken, and as no mention is made in the paper of tests on smooth-faced ogee spillway dams, it appears that such a dam was not seriously considered owing, no doubt, to the apparent difficulty of properly absorbing the high velocities of the water at the toes of dams of that type. In the light of knowledge gained during the past ten years, the apparent difficulty has disappeared, and as there are certain inherent serious disadvantages in the stepped type of overfall dam, there are strong reasons for believing that a series of experiments as ably conducted should have been carried out to determine physically the feasibility of a smooth-faced ogee dam before the stepped profile was adopted.

The writer has never been enthusiastic over the stepped type of spillway dam, notwithstanding its occasional use in the northeastern part of the United States. It is not easy for him to get away from the idea that the absorption of hundreds of thousands and possibly millions of horse-power of energy by "pounding it out" on the dam itself cannot be done without immediate serious destructive action on the dam and within areas where such action is least desirable and most dangerous.

Cyclopean and coursed masonry and even concrete deteriorate considerably in time in so far as freedom from cracks is concerned. Wide variations in temperatures and to some degree in stresses, due to loads, ultimately develop cracks, possibly very minute but, nevertheless, present and extending to considerable depths. This tendency to crack is considerably facilitated by the greater exposure and angles introduced by the stepped construction and by the entrance of water under direct impact, which takes place on every step. It can be only a relatively short time before the corners of the steps are broken away, due to weathering and impact. After this has happened, it is highly probable that the exceptional flood will come and the steps, now no longer of sufficient width to function properly, will then become a source of positive danger. In the same manner as discussed in the experiments, where the steps were not made sufficiently wide, some of the water with its velocity unbroken will reach the lower parts of the face of the dam with sufficient energy to develop erosive action which the lower steps cannot withstand for any appreciable length of time. The result will be rapid destruction of the steps and serious attrition of the main body of the dam near its base where the stresses are greatest and where adequate repair will be most difficult if not impossible.

The great rapidity with which raveling action proceeds where water is flowing at high velocities over or directed against checked and cracked sur-

* Asst. Engr., Aluminum Co. of America, Pittsburgh, Pa.

faces, especially where these surfaces are irregular, is familiar to many engineers. The writer has in mind as a particular case the tremendously destructive action of water flowing at a velocity of about 45 ft. per sec. in a spillway channel newly excavated in a very hard rhyolite formation. The surface as left by the contractor was rough and broken, but as blasting had been very light in order to prevent as far as practicable the breaking of the rock left in place, it is difficult to believe that fracture at depths greater than 10 ft. could have been more than what one would term "hair cracks", yet in the course of a few days during the first big flood discharging through the channel, holes 15 ft. or more in depth were scooped out of the ledges in place and blocks of rock many tons in weight carried down the channel. The average slope of the channel was about 30 ft. vertical in 100 ft. horizontal, and the maximum depth of water in the channel at the beginning of the flood was about 7 ft.

Water may be allowed to flow over smooth concrete at very high velocities for a long time without appreciable wear, but where slight irregularities develop, erosive action appears and increases with great rapidity. The initial irregularities need be very slight to start the trouble. The writer has recently inspected some photographs of the down-stream toe of an ogee spillway dam, over which water has been flowing with velocities of more than 100 ft. per sec., in which there is evidence that wear is beginning at the horizontal joints marking different pourings of concrete, and it is safe to say that the erosion at these points will develop rapidly. These bonding joints were made with the greatest care and the lines of division must have been microscopic, nevertheless, they appear to be sufficient to give the water its initial "grip". On the adjacent areas, originally smooth and unbroken, the wear is very slight.

The statement no doubt will be made that the steps can be repaired and renewed from time to time, but this involves a heavy maintenance charge and if not done may ultimately result in trouble, as it is not certain that those in charge of the dam in the future will appreciate the necessity of keeping these steps in perfect repair. It is doubtful whether the repaired sections would have the length of life of the originals, as it is extremely difficult to bond new and old concrete or masonry in such a manner that water flowing at high velocity will not soon begin to erode the joints. Furthermore, the filling of holes with concrete so that microscopic joints can be avoided is practically an impossibility, owing to the tendency of the filling concrete to shrink away from the hole in which it is cast.

As previously stated, during the last few years much knowledge has been acquired regarding the conditions under which high velocities can be absorbed successfully or controlled by water-cushions and the burden of acting as a buffer shifted from the dam to areas more remote from the vulnerable parts of the structure and to construction more readily and satisfactorily repaired when the inevitable wear makes such action necessary.

Early in 1914, the writer performed a fairly elaborate series of experiments on models of an overflow dam to be constructed on the Little Tennessee

River, about forty-five miles south of Knoxville, Tenn. The prototype was to be 195 ft. high and the models were on a scale of 1:48. The conclusions drawn from the experiments were summarized in a report made to the Knoxville Power Company, in June, 1914, the more essential paragraphs of the conclusions being as follows:

"Practically all of the kinetic energy of water flowing at high velocities in open channels may be internally absorbed by directing the water into pools of water exceeding a certain minimum depth and a certain minimum length.

"The theoretical minimum depth of the pool is given by the equation: $d_2 = \sqrt{\frac{d_1 V_1^2}{2g}}$, in which d_1 is the depth of the water before entering the pool and V_1 is the velocity of the water at the same point. This equation gives the required depth of the pool, d_2, above the same horizontal datum above which d_1 is measured.

"The required length of the pool is at least approximately equal to the net velocity head absorbed by expansion divided by 1.35. This gives the distance in which the bottom velocity is completely absorbed. About 20 per cent. should be added to allow for complete absorption of velocities above the bottom.

"In high dams, subject to high velocities at the toe, complete absorption of the high velocities within the shortest distance possible is desirable to prevent channel erosion and turbulent conditions in tail-races, as much as possible.

"An efficient method of absorption is by directing the water into pools having the minimum depth given in preceding paragraphs.

"The height of the lip of the bucket above the bottom of the pool has little effect on the general efficiency of the absorption if the lip is kept within the lower half of the pool's depth. The bucket with the lowest possible lip will allow the least wear on the concrete, but as this is negligible on a well designed and constructed dam, the advantages of the highest possible bucket in decreasing the height through which the water falls and also allowing the use of longer buckets along the axis of the dam with less excavation and less concrete may be considered paramount. The highest possible position of the lip with present knowledge is perhaps about half the depth of the pool.

"The required depth of pool varies in the same sense as the depth of water on the crest of the dam, hence the longer the crest the shallower the required depth of the pool. The bucket must of course also be made longer.

"For the same head on the crest, arched dams, by increasing the value of d_1 for the same discharge, require greater depths of cushion pools than straight dams.

"Where pools of sufficient depth are not provided, it is probable that the entire kinetic energy of the water will be expended upon the channel and erosion will continue at a rapid rate until a pool of sufficient depth is excavated by the water."

During 1916, and, subsequently, the Miami Conservancy District made elaborate experiments that clearly demonstrated the efficiency of the hydraulic jump as a means of absorbing high velocities. The data and conclusions were published as Part III of Technical Reports, under the title, "Theory of the Hydraulic Jump and Backwater Curves and the Hydraulic Jump as a Means of Dissipating Energy". Mr. R. D. Johnson, in his discussion* of the paper, by Karl R. Kennison, M. Am. Soc. C. E., entitled "The Hydraulic Jump, in

* *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 378.

Open-Channel Flow at High Velocity", and in a paper* on "The Correlation of Momentum and Energy Changes in Steady Flow with Varying Velocity and the Application of the Former to Problems of Unsteady Flow on Surges in Open Channels", presents in an excellent manner the mathematical and physical conceptions involved in the hydraulic jump and the related hydraulic feature, the standing wave, and casts aside the shroud of mystery with which the every-day engineering world seems to have covered this really common and natural occurrence. It is probable that the theory of the hydraulic jump and its practical application are now fairly well understood and that few dams of any magnitude are constructed without consideration of its development and whether or not artificial means of insuring the jump will be required now or in the future.

The writer considers the ideal spillway dam to consist of a perfectly smooth-faced ogee dam, straight in plan, with the crest unbroken and designed to conform to the lower nappe of the falling water for the highest possible flood; the down-stream face of the dam to be joined to the floor of the channel by an easy curve; the channel down stream to be the same width as the crest of the dam for a distance sufficient to permit the full development of the normal jump (farther down stream the channel may be varied to suit conditions stated subsequently); the floor of the channel near the dam to be nearly level throughout, and as its position vertically is defined by the low point in the natural channel, this requirement also agrees with the one that the water should be taken through the entire fall in a single drop along the entire crest. Finally, the hydraulic functions of the channel down stream from the dam should be such as to hold the jump at the toe of the dam. Natural conditions may suffice; if not, secondary dams may be used, or the channel may be deepened artificially or be permitted to deepen naturally, as it will if sufficient depth is not provided originally.

To develop the ideal spillway economically, the dam would have to be built in a U-shaped valley. This is rarely possible, especially where very high dams are involved, the valley usually being V-shaped with respect to the dam. To meet as nearly as possible the ideal requirements, economical construction then suggests widening a certain part of the bottom of the V to permit the dropping of as much as possible of the water for the full height to provide for frequent floods and placing gates on the crest of the dam to hold the flow as much as possible within the length of crest discharging its waters directly into the cushion pool. The remainder of the crest is to be opened to flow only as required. The stepped profile may be used in these sections, or if the foundation rock is of such quality as to resist erosion well, the water from an ogee spillway may be turned over it. In either case, however, repair work from time to time will be necessary.

If gates are not permissible on the crest, a rather difficult problem is presented. If water is spilled for a considerable part of each year, the best solution seems to be in smoothing the rock surface and armoring it with concrete, the lower end of the slope to be treated the same as an extension of the dam

* *Engineers and Engineering*, July, 1922.

and the water guided horizontally into cushion pools by easy vertical curves. In any case, the direct impact of water at velocities in excess of, say, 10 ft. per sec., is to be avoided and all the water guided by surfaces made as smooth as possible into ample cushion pools.

In the cushion pool itself, very little armoring is required, as the energy is absorbed entirely by the water. Mr. Johnson has raised the interesting point that under ideal conditions in the hydraulic jump there probably is a certain quantity of water "humped up" at the bottom of the jump to form a natural weir. The inference is then that this water is practically "dead". If this is true, then high velocities are absent immediately above the channel floor and, therefore, erosion on surfaces on which the jump actually takes place is practically absent. This means that if the initial point of the jump is held on the concrete at the toe of the dam, erosion down stream from this point need not be feared, as long as the surface of the bucket and the floor of the channel are at the same elevation and the jump is actually formed. The writer is inclined to agree with this proposition. His own experiments were not sufficiently detailed to give any information on this subject. A few experiments recorded by the Miami Conservancy District, shown on page 79 of the published report, seems to confirm Mr. Johnson's theory. Several of the writer's experiments were made on models on which the horizontal edge of the bucket was about 2 in. above the floor of the channel. A favorite test was to fill the channel with sand level with the edge of the bucket and then start the water over the model after making sure that the jump would be developed. Under uniform flow conditions, the sand for several inches down stream from the bucket would not be moved, although velocities of 14 ft. per sec. prevailed within 1 in. of the edge of the bucket. Eddies, however, could be produced by slight manipulations that would sweep out all the sand and, therefore, the experiments were not conclusive.

With regard to the Gilboa Dam, it would be admittedly somewhat difficult to utilize readily the hydraulic jump throughout the entire length of structure, but it appears from such data as are available that at least two-thirds of the dam could have been built as a smooth-faced ogee spillway without difficulty and the water from this part of the crest directed into a cushion pool. It is reasonably certain that a far more efficient method of absorbing the high overfall velocities would have been utilized and the chances of future trouble greatly minimized. With a carefully designed crest, using the same free-board specified for the dam actually being built, the crest length for a maximum discharge of 110 000 sec-ft. could have been reduced to 900 ft., assuming a weir coefficient of 3.90 which is reasonable. At the maximum section, the discharge would be 121.5 sec-ft. per foot of crest, the velocity of the water at the toe of the dam about 98 ft. per sec. and the depth, at the section at which this velocity exists, 1.24 ft. According to Unwin's formula for the jump, the required depth of water to develop the jump for these conditions would be 24.8 ft. per sec., and the writer is inclined to believe that the natural flow conditions down stream from the dam would easily maintain this depth. If not, a low dam not less than 140 ft. from the main dam would be required.

Owing to the much higher elevation of the natural ground at the base of the dam toward its ends, not more than 600 ft. of the shortened crest could be protected economically by the main cushion pool. The discharge for the remaining 300 ft. of crest would have to pass into cushion pools set step-fashion at higher levels and of much more expensive construction, or down smoothed and armored slopes into the main pool, or the velocity of the water destroyed by use of a stepped profile for the dam, the actual method to be used to be developed by comparative studies.

The writer hopes to publish data on the conditions at the toes of two overfall dams 200 ft. high, completed in 1917 and 1919, respectively, which have been subjected to several floods since their completion. In certain parts and at certain times, natural water depths have not permitted the jump to develop, and high velocities have occurred resulting in considerable erosion. In time, natural cushion pools will be formed below these dams, and reasonable protection will then be obtained against continued erosive action without secondary dams. In any case, he hopes more information will be derived on the interesting problem of endowing high overfall dams with the longest possible life.

THADDEUS MERRIMAN,* M. AM. SOC. C. E. (by letter).—The writer has long been convinced that the use of accurate scale models in the solution of hydraulic problems involving dynamic effects will give better results than can possibly be secured by abstract theoretical considerations. During the greater part of the work so well described by the authors, he was in close touch with the experiments conducted by them and will refer briefly to the cause which he believes to be responsible for the dissimilarity referred to in the paper as having been observed in the behavior of models of different scales.

Water on approaching a weir crest has one free surface; but immediately after passing the crest, it has two such surfaces. For the same length of weir, irrespective of its crest width and of the depth of flow, the increase of free surface will be the same. As surface tension as between water and air is a constant for each unit of area, it follows that, when the surface is suddenly doubled, energy must be expended. The amount of energy expended in the formation of the under side of the nappe is proportional to its surface area and as this area, in turn, is proportional only to the weir length, it appears that the energy required for the formation of the under side of the nappe is not proportional to the scale of the model. It follows as a necessary consequence that models of different scales will not exhibit the same characteristics for all depths of flow over them. The thinner the sheet the greater will be the divergence from true similarity of behavior. This difference is clearly shown in Fig. 13, where the divergence for the proportionate depth of 3 ft. is materially greater than that for the proportionate depth of 6 ft.

The differences observed are due to the fact that the energy required to form the under side of the sheet is a constant for each unit of crest length. The thinner the sheet the greater will be the ratio between the energy required for its formation and the total energy in the vertical plane of the weir crest;

* Chf. Engr., Board of Water Supply, City of New York, New York City.

that is, the energy remaining per unit volume of the sheet, after the nappe has been formed will be least for the thinnest sheet, and, in consequence, the thinner the sheet, the less far it will be projected. Approaching the limit of zero thickness the nappe, instead of being projected beyond the crest, would either fall straight downward or be pulled inside the vertical. This is a phenomenon which the writer has often observed, but which is much complicated by the surface tension as between the material of the weir crest and the water which usually has a different value from that between the water and the air.

This factor of surface tension is noticeable only at shallow depths of flow, and it is only at such depths that marked dissimilarity is observed. The differences which were noted at greater depths are those naturally incident to errors of observation, which include not only the personal equation of the observer, but also differences due to changes in the direction and intensity of the wind as well as to errors in the setting of the measuring apparatus which for any one series are generally constant.

It is of interest to note that the value of the surface tension as quoted by Besant* is 7.83 mg. per mm. of length. This tension, if multiplied by the rate at which the nappe is being formed, will represent the energy required for the formation of the sheet per millimeter of its length. This energy is a small proportion of the total energy of the system and for any appreciable thickness of the nappe is entirely negligible. At low heads, however, its effect becomes noticeable.

R. W. Gausmann,† M. Am. Soc. C. E., and C. M. Madden,‡ Assoc. M. Am. Soc. C. E. (by letter).—The writers are pleased to note the valuable discussion on this paper. It seems that all the discussors agree that models in whatever sense the word is interpreted, if properly used, may give valuable information for the preparation of designs of engineering work and, more particularly, for hydraulic structures.

In the early stages of the experiment, some doubt was expressed as to the usefulness of these models, and this doubt was intensified when it was found that models of different scales did not give identical results. It was only after many experiments with models of different scales that the writers felt safe in predicting that the behavior of the full-scale prototype would approximate the behavior of a small model.

The reference made by Mr. Honness, to the experimental models of the Boonton Dam, Cross River Dam, and Ashokan Reservoir Waste Weir are very interesting, as they show, particularly in the first two instances, that models have been used successfully in the development of hydraulic structures.

Mr. Ewald has discussed certain points that were not brought out in the paper. Experience has led the writers to believe that in this climate there is a tendency for exposed surfaces of concrete to disintegrate. All masonry surfaces, therefore, are to be faced with stone indigenous to this region, which is not susceptible to weathering. The floor of the spillway channels also is to be paved with sandstone and grouted.

* "Elementary Hydrostatics", Cambridge, 1892.

† Div. Engr., New York City Board of Water Supply, Allaben, N. Y.

‡ Asst. Engr., Board of Water Supply, New York City, Cold Spring, N. Y.

On the dam, the edges of the steps are formed with stone from 6 to 36 in. in thickness, set on edge. These stones are 3 ft. deep (vertical) and from 3 to 5 ft. long (horizontal). As an added precaution, anchors not less than 5 ft. apart secure these edge stones to the cyclopean core. The treads are paved with stones from 3 to 36 in. and more in thickness (horizontal), an average depth of 21 in. (no stone is less than 18 in.), and varying lengths which must not exceed five times the thickness. The risers are faced with similar stones, except that the average width (horizontal) is 17 in. All the joints are carefully grouted and, later, will be pointed. An excellent quarry has been opened near the Gilboa Dam from which large blocks of blue-gray sandstone are readily obtainable. The minimum thickness of the stone placed to date is seldom less than 12 in. It is thought that this surface will be less subject to erosion and disintegration than concrete.

It will be noted from Fig. 4 that the grade of the channel is not excessive. The grade of the upper third is approximately 12.5%; the middle, 6.1%, and the lower third, 4.6 per cent. This diagram also shows that there is a considerable depth of water. In the part of the channel in the old stream bed, which is at the foot of the highest part of the dam, there is a depth of about 30 ft., due to a head of 6 ft. (163 cu. ft. per sec. per sq. mile). This depth due, in part, to the reconstruction of an old power dam, should provide an efficient water-cushion.

It is hoped that the theory of models may be developed more fully. At present, it seems as if practical difficulties would be encountered in making diminutive structures which would represent in detail the performances of their large-scale prototypes, but it is possible that careful study might eliminate these difficulties and that it would be possible to reproduce in miniature, the destructive effect of Niagara Falls or the effects of the propelling charge of a 16-in. gun.

In conclusion, the writers wish to express their thanks to those who so carefully discussed this paper and also their appreciation for the assistance and encouragement given by those who either directed or assisted in this work, more particularly to J. Waldo Smith, M. Am. Soc. C. E., Chief Engineer until July, 1922, Mr. Merriman, the present Chief Engineer, Mr. Honness, Department Engineer, and James A. Guttridge, M. Am. Soc. C. E., Division Engineer, all of the New York Board of Water Supply.

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THE DESIGN OF STRUCTURAL SUPPORTS FOR TURBO-GENERATORS*

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Synopsis

The modern turbo-generator is a highly complicated machine, subject, at times, to intricate vibrations and to forces which may attain great magnitude, owing to the high operating speed. Its structural pedestal, of concrete or steel, must provide a substantial support, both during normal operation and at times of disturbance of normal conditions.

The design of such a pedestal presents many interesting problems to the civil engineer. The restraint of vibrations and the restriction of deflections to very small amounts are the controlling factors throughout the design.

In this paper, the relative merits of the three common types of pedestals, reinforced concrete, structural steel, and composite, are discussed. A brief outline is given of the nature of the vibrations, and the suggestion is made that, in addition to the vertical impact allowance now generally used, a definite horizontal impact allowance should be adopted.

In conclusion, it is suggested that utility or fitness, rather than relative cost, should be made the sole basis of comparison in deciding on the type of pedestal to adopt for a turbo-generator installation.

When a reciprocating engine and generator is replaced by a modern turbo-generator, the operating engineer is always impressed with the marked economy

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of space and weight attained. The civil engineer who is called on to design a structural support for the new unit, is equally impressed with the saving in weight and space, when he compares the new foundation with that which previously served. In many instances, it is found practicable to utilize the sub-foundations of old engines for the support of modern units of many times the power of the machine replaced.

The introduction of the turbine-driven electric generator has caused the evolution of a distinct type of foundation. Where previously the support for the engine and generator consisted of a large sub-foundation and a massive superstructure, it is found possible, with the turbo-generator, to mount the unit on a comparatively light and open pedestal, with space in the basement below the machine for the condenser, circulating pump, and other auxiliaries. The absence of the unbalanced thrusts on the foundation, which occur with the reciprocating engine, has rendered this type of construction possible.

The fact that turbo-generators have only a fraction of the weight of reciprocating units of equal power, together with the freedom from the great impacts of the latter type, resulted, at first, in a tendency to slight the subject of turbine foundations. This is noticeable in some of the older reference books on power-plant design. It is now generally recognized that special problems occur in the design of such foundations, and that great care is necessary if an adequate and efficient support is to be obtained.

The generator with its prime mover is, perhaps, the most important element in the whole power plant, and principles of false economy should not be allowed to result in an inadequate supporting structure for it. The design should be made by an experienced engineer who realizes that it is much more than a problem of sustaining static loads and who is familiar with the dynamic effects of the revolving machine during normal operation and at times of accident. This supporting structure may be considered as consisting of two elements, namely, the sub-foundation below the basement floor or condenser-pit level, which may be called the "mat" or "footing", and the steel or concrete structure extending from this level to the base of the machine, which is often referred to as the "pedestal."

The type of condenser selected for the unit is a controlling element in deciding on the type and proportions of the pedestal. The value of a high vacuum was early recognized in the development of the steam turbine, and much study has been given to the construction of condensers to attain this end. At the same time, the ill effect of long piping and bends in the line between the condenser and the turbine exhaust has been emphasized, and a type of installation has been developed in which the condenser is mounted directly beneath the turbine with a resulting increase in efficiency. When financial and operating considerations result in the selection of a jet condenser for the unit, the problem of the design of the pedestal is much simplified, as there is usually ample space for the pedestal beams, columns, brackets, and bracing. The selection of a surface condenser, however, often makes necessary considerable ingenuity on the part of the structural designer to obtain a proper pedestal without interference of these members with the condensing unit.

Three types of pedestals have been evolved, as follows: (*a*) the all reinforced concrete type; (*b*) the structural steel type; and, (*c*) the combination structural steel and concrete, or composite, type.

The reinforced concrete pedestal is probably the most common type of turbo-generator support. The main advantage of this type lies in the relatively great volume of concrete in the beams and piers which have sufficient mass to absorb any vibrations that may develop in the machine, and, with reasonable care in design and construction, satisfactory operating conditions, as far as vibration in the pedestal is concerned, are practically assured. This pedestal has, however, several distinct disadvantages. The massive concrete members which are necessary to carry the loads, often seriously restrict the space required for the condenser, the ventilating air-ducts, and the piping. It is often quite difficult to secure as compact an arrangement for the condensing and auxiliary equipment as is desirable and, in general, the space below the unit for operation and dismantling will be more limited than with a steel pedestal. The weight of a concrete pedestal is several times as great as that of a steel pedestal. This results in a more expensive sub-foundation or mat, and may present a serious problem where subsoil conditions are poor.

These disadvantages have resulted in the development of the structural steel pedestal and the combination steel and concrete type, often called the composite pedestal. No sharp line of demarcation exists between steel and composite pedestals. The latter type, as generally understood, has its main members of structural steel with concrete filling between girders, a concrete deck, and, in some cases, concrete encasement of all steel members. The disadvantages noted for the concrete pedestal are largely overcome if a steel or composite design is adopted, and, it is much easier to provide members of adequate strength, at points where clearances are most restricted.

It is often necessary to defer the completion of the erection of the pedestal until the condenser has been placed, due to the space needed for erecting the latter. In such cases, the construction schedule for the plant and equipment depends on the date of the erection of the condenser. Once the condenser is placed in position, it becomes desirable to complete the erection of the pedestal as soon as possible, in order to be ready for the generator. In such cases, the fact that less time is necessary to construct a steel pedestal than one of concrete, is an argument in favor of adopting the former type.

The relative space occupied by concrete and structural steel pedestals is illustrated in Fig. 1, which shows a typical concrete design suggested by one manufacturer and a steel design for a unit of the same capacity recommended by another manufacturer. The roominess of the steel pedestal is apparent.

The tendency of the steel pedestal to vibrate with the machine constitutes its one disadvantage when compared with one of concrete, and the design of a steel pedestal proportioned so as to avoid this vibration is a difficult problem. The tendency to vibrate exists in all turbo-generators. The unit is a complex machine operating at high speed, and vibratory forces and moments may exist, due to many causes. Normally, the vibrations are restrained to negligible amplitudes by care in the manufacture of the unit and by proper precautions on the part of the operator.

Vibrations may be due to excessive bearing clearances, deflection of the spindle, disturbance of clearances at various points producing rubbing, lack of alignment, or to unbalance in the rotor caused by erosion of blades, incrustations on blades, breaking of buckets, etc. Extreme care is taken by manufacturers to correct all unbalance in the machine, and ordinarily it leaves the shop in practically perfect balance. Assuming that it is properly installed, the principal factors which will tend to produce vibrations are those which may develop in the course of its continued operation in the form of one or another of the causes stated.

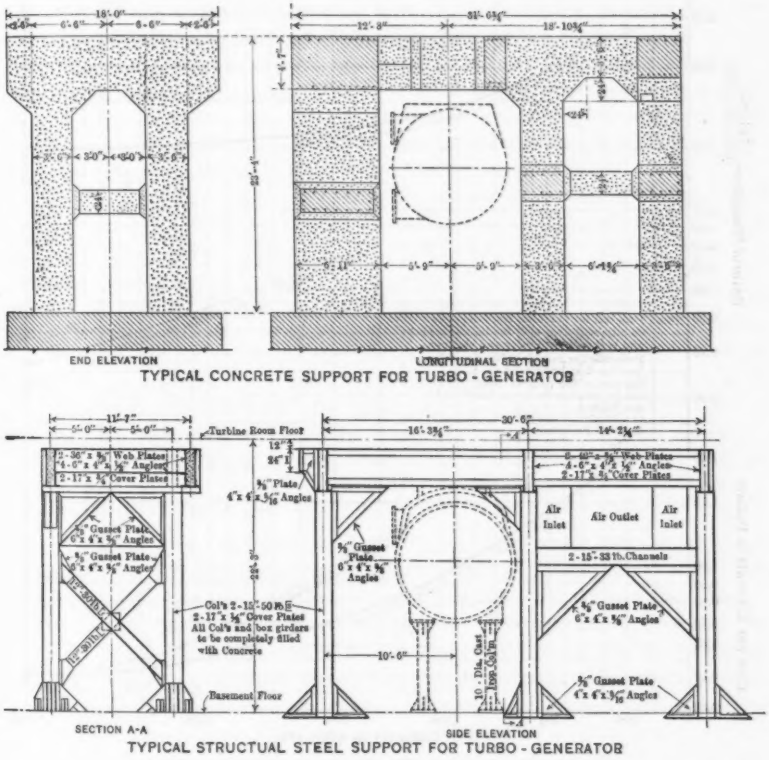


FIG. 1.

Unbalance of the rotor may occur in either or both of two forms. Static unbalance will exist when the center of gravity of the revolving parts does not lie in the geometric center of the shaft. Dynamic unbalance will occur if an unbalanced couple exists in any plane containing the shaft axis. Thus, a machine may be in static balance, but be unbalanced dynamically. The high speed of the turbine renders unbalance very serious as an unbalanced mass of 1 oz. will generate a centrifugal force of thousands of pounds.

Vibrations in the turbo-generator, from whatever cause, will be transmitted to a greater or less extent to the pedestal. If the latter is of a light, non-rigid construction, the vibrations will be amplified, resulting in serious damage to the unit. The pedestal must have mass enough to absorb all vibrations and be rigid enough to prevent swaying. If it is of great mass, the time necessary to overcome its inertia will be sufficient to consume the fractional part of a second elapsing until an impact in the opposite direction is applied, and its vibrations will be restrained to negligible amplitudes.

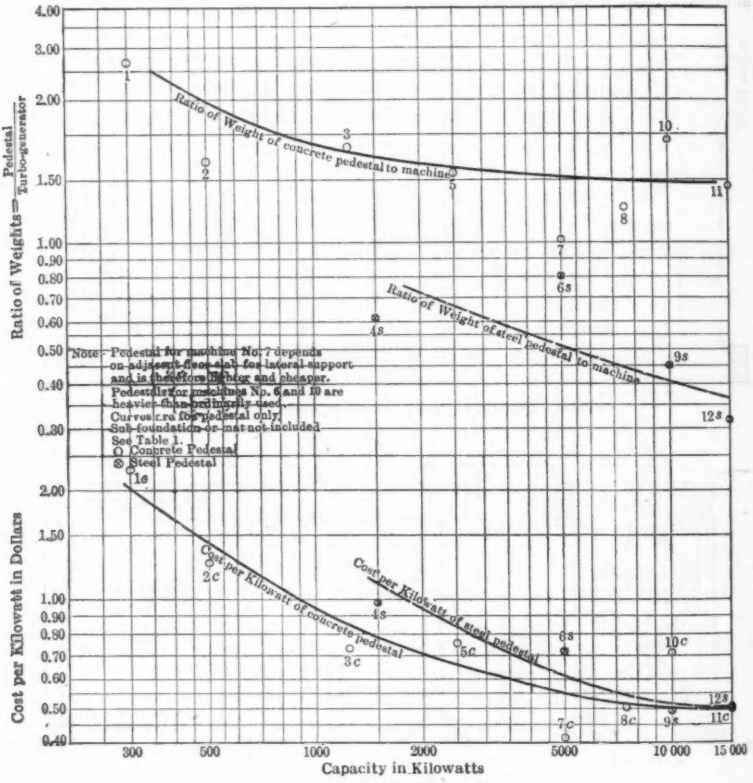


FIG. 2.

The most practical means of arriving at the requisite mass for a turbo-generator pedestal is by a study of existing designs in satisfactory operation, both as to the total mass and its distribution. In this connection, the weight-ratio curves of Fig. 2 are of interest. Each of the points plotted represents the ratio of the weight of the complete pedestal (excluding the sub-foundation or mat) to the weight of the unit supported, covering the range from 300 to 15 000 kw. Existing installations and, in some cases, typical manufacturers'

designs, have been used. The curves (Fig. 2), indicate that concrete designs, as now made, weigh from two to four times as much as steel designs. For both types, the weight ratio decreases as the units become larger.

It is not intended to convey the impression that the weight-ratio and cost curves shown in Fig. 2 will give the precise weight or cost of any pedestal that may be considered. The sources of information used are too diverse to permit of this, and the factors entering into the design are too variable to allow of such an interpretation. The only way to secure such curves would be to design pedestals over the whole range covered, with identical assumptions as to impact allowances, condensers, etc., and to compute the weight and cost of each. If this is done, it is possible that some definite law as to the variation of such factors can be deduced. The writer's intention, however, is to convey only a general idea as to the average variation of costs and weights for present designs of various capacities. It is believed that the curves represent average practice in pedestal design, as they are constructed from data obtained from installations in satisfactory operation, and, in several instances, from typical designs recommended by turbo-generator manufacturers.

It is possible to conceive of a pedestal possessing great mass, but supported on unbraced stilt-like columns so as to allow sway or oscillations of large amplitude, but of low frequency. In order to prevent this, it is necessary to introduce bracing and to make the columns capable of resisting bending. To determine the size of these members, the conception of the vibrating machine must be translated into one of equivalent forces, or specifically into horizontal and vertical forces acting at the top of the pedestal. These forces will be roughly proportional to the mass of the turbo-generator and to its acceleration in the direction of vibration. By making certain assumptions as to the amplitude of the vibrations and their frequency, it is possible to derive an expression giving the relation between the weight of the turbo-generator and the force that it exerts in the direction of vibration or sway, thus, $F = \frac{K}{100} W$. In determining K, the writer believes that practical considerations should govern, such considerations being based on design assumptions which have been found to give satisfaction in existing installations. It is certain that K will not vary directly with the speed, as it will have a maximum value at the critical speed and will be much less at higher and lower speeds. Other indefinite factors governing K are changes in the pressure of the steam, temperature variations, and changes which may occur in the condition of balance.

This impact allowance, K, is the percentage dead weight of the machine acting as transverse horizontal and vertical forces at the top of the pedestal. There is a great diversity of opinion as to the proper impact allowances. This is due partly to the failure specifically to define the allowable unit stresses, as will be noted later, and often to ignoring the horizontal impact allowance altogether. In some cases, the practice seems to have been to adopt a high vertical impact allowance, together with small working stresses, and to ignore the horizontal forces which exist. It appears more logical to assume definite horizontal and vertical forces, in addition to the dead weight of the machine, and

to adopt working stresses more nearly equal to those used in general structural design. For ordinary designs, the writer has used horizontal and vertical impact allowances varying from 33⅓ to 50% of the dead weight of the machine.

Certain general principles exist relative to the proper proportions of the pedestal to resist best the horizontal impact force noted previously. These principles can best be considered by an examination of the pedestal frame as regards vibrations. Every pedestal has a "period of free vibration", that is, a definite time which would elapse during one complete oscillation, if it was sprung from its normal position and left free to oscillate without receiving further exterior impulses. This free period of the pedestal is dependent on the size, length, and arrangement of its members and on the material of which they are constructed. Complex computations would be necessary to determine just what would be the "free period" of any one type of steel pedestal, and uncertainties as to the effect of the joints would render such computations of little value. The more rigid a pedestal is made, the shorter its natural period will be, and its members should be arranged to secure this end.

Fig. 3 represents an end view of a typical pedestal. This frame will have the shortest natural period of vibration, and, therefore, maximum rigidity, when the center of gravity of the turbo-generator and pedestal is made as low as possible, and when the ratio of $\frac{H}{W}$ is made as small as clearances and available space will permit. In some installations, a low center of gravity is obtained by hanging the condenser from the top of the pedestal and thus utilizing its mass in addition to that of the turbo-generator and pedestal. Table 1 gives the ratio, $\frac{H}{W}$, for several designs noted by the writer, together with other pedestal data. The machine numbers correspond to those used in constructing the curves of Fig 2.

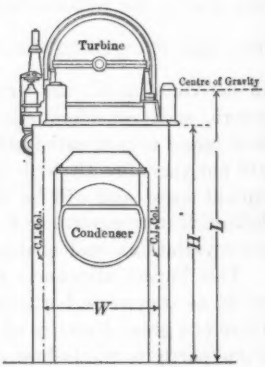


FIG. 3.

It should be stated that some designers consider it possible to have a pedestal of too great rigidity, although it is believed that all turbo-generator manufacturers concur in the desire that they be made very rigid. Comparison has been made between units resting on very rigid pedestals and instances of lathes and similar machines which have suffered from vibrations when supported directly on a concrete base. A design of a turbo-generator pedestal has been developed by Mr. N. W. Akimoff to overcome this objection of too great rigidity. In this design, the unit rests on a three-point support with heavy locomotive springs between it and the pedestal to absorb shocks. The writer understands that no pedestals of this design have been installed.

If the natural frequency of vibration of the pedestal or of any of its members equals the speed of the machine, resonance occurs. Successive impacts

will build up the kinetic energy of the vibrating pedestal, and the amplitude of the vibrations will increase until a balance is reached between the energy of each impact and that of the restraining forces, due to the stiffness of the pedestal, or of the member under consideration. Thus, resonance will result in high stresses in the pedestal.

TABLE 1.—TABULATION OF PEDESTAL DATA

Machine number.	Pedestal.	Capacity, in kilowatts.	Condenser.	Ratio of weight of pedestal to weight of turbo-generator.	PEDESTAL:				
					Weight per kilowatt, in pounds.	Cost per kilowatt.	H.	W.	$\frac{H}{W}$
1.....	Concrete	300	.....	3.17	195.0	$2.30	6.6	4.0	1.65
2.....	Concrete	500	Jet	1.69	105.4	1.25	10.0	4.25	2.35
3.....	Concrete	1 250	Surface	1.84	62.0	0.73	7.0	5.30	1.32
4.....	Steel	1 500	Surface	0.61	18.0	0.99	17.75	8.33	2.13
5.....	Concrete	2 500	Surface	1.58	48.6	0.76	17.5	6.46	2.71
6.....	Steel	5 000	Surface	0.82	22.5	0.72	18.0	9.5	1.90
7.....	Concrete	5 000	Jet	1.02	27.9	0.41	19.21	8.29	2.35
8.....	Concrete	7 500	Jet	1.26	33.5	0.50	23.30	8.0	2.92
9.....	Steel	10 000	Surface	0.46	13.9	0.49	22.25	11.58	1.92
								10.0	2.23
10.....	Concrete	10 000	Surface	1.91	57.4	0.71	23.38	9.5	2.45
11.....	Concrete	15 000	Jet	1.45	44.2	0.49	20.0	10.5	1.9
								12.0	1.67
12.....	Steel	15 000	Surface	0.32	9.3	0.51	20.0	18.0	1.17

A quantitative determination of vibrations induced in a turbo-generator due to non-rigidity of the pedestal has been noted.* In this instance, serious vertical vibrations in the generator bearing of a 10 000-kw. unit were detected and measured by a vibrograph. The supporting beam under the bearing and its columns were reinforced, and it was found that the vibrations were reduced to safe amplitudes.

If the speed of a turbo-generator is varied, it is observed that a certain definite speed exists at which vibrations in the unit are quite marked, although it may run quite smoothly at all other velocities. This speed at which the vibrations are so noticeable is called the "critical velocity." This phenomenon is due to the bending of the shaft under the centrifugal forces generated by unbalance, and electrical and mechanical engineers are familiar with the theories and experiments of DeLaval, Stodola, and others, demonstrating the mechanical principles involved.

The common working speeds of turbo-generators are 1 500, 1 800, and 3 600 rev. per min., depending on the cycle and number of poles. In general, the critical speed of units as now constructed is greater than the two lower normal speeds noted. However, for machines of 3 600 rev. per min., the critical speed is likely to be below the working speed, and in such cases, in starting up and shutting down, it must be passed through. Although manufacturers make special provisions in machines operating above the critical speed, it is well for the pedestal designer to make a more generous impact allowance for high-speed machines.

* *Mechanical Engineering*, August, 1922.

In addition to the transverse horizontal forces which have been considered, vibrations and impacts may exist in a direction parallel to the axis of the shaft. The unit is self-contained as far as axial forces due to unbalanced steam flow is concerned, as the thrust-bearings transmit these forces to the casing where they are balanced by the steam pressure in the opposite direction. Vibratory vertical impacts, however, will produce axial impacts if any eccentricity exists between a bearing and the pedestal member which supports it, and, in general, it is found necessary to provide longitudinal bracing as substantial as the transverse bracing.

Another source of shocks which may come on the pedestal, lies in the disturbance of the magnetic torque of the generator, which takes place during short circuits. Under normal operating conditions, the magnetic torque need not be considered, as far as the pedestal is concerned, but short circuits will cause violent fluctuations of this torque, and the pedestal will receive serious impacts.

TABLE 2.

Force.	Direction.	Point of application.
Weight of generator	Vertical	Load points given on manufacturers' plans.
" " turbine	"	
" " bed-plate	"	
Dead weight of members	"	.....
Vertical impact allowance	"	.....
Transverse impact allowance	Horizontal	At center line of shaft.
Condenser and water content	Vertical	Dependent on method of carrying condenser.
Vacuum load	"	
Axial forces	Horizontal	At center line of shaft.

The following steps to be pursued in the design of a pedestal, are taken from standards used by the writer in the structural designs of power plants. (See Table 2.) No specific directions are attempted, as there is such a wide variation in loads, load points, construction of units, etc., for turbo-generators of different capacities and manufacture, but certain general steps are outlined for the guidance of structural designers engaged in the work, as follows:

1.—Decide on the probable type to be used: Concrete, structural steel, or composite.

2.—Preliminary layout, working from certified prints of equipment, piping, and electrical plans, to determine the best arrangement for beams, columns, and bracing, using assumed sizes. At this time, the piping and electrical layouts should be advanced sufficiently to indicate the necessary clearances for all elements adjacent to the pedestal. Investigate the approximate limits for H and $\frac{H}{W}$. H will be determined by the condenser size, floor levels, and crane clearance.

3.—Design the pedestal members in the following order, in which a 5 000-kw., Westinghouse, semi-double flow unit has been taken as an example:

(*a*) Design the slab and secondary members at the top of the pedestal.

(*b*) Compute R_1, R_2, R_3, M_1, M_2, M_3, and the shears for the main longitudinal girders; compute the average of the bending moment curves, *A* and *B*, Columns 1 and 3 (Fig. 4), to take the moments, $\frac{M_1}{2}$ and $\frac{M_3}{2}$, respectively.

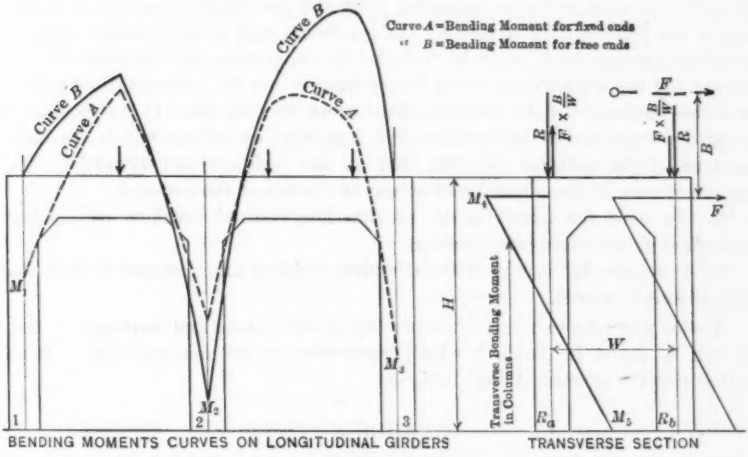


FIG. 4.

For a steel design, select by trial a beam to give a deflection, in either span, of less than 0.02 in. For a tentative value, let $f = 5\,000$ lb. per sq. in.

For a concrete design, select a beam for bending, $f_c = 600$ lb. per sq. in. Compute the deflection, maximum $d = 0.02$ in.

(*c*) Investigate the transverse bents, Nos. 1, 2, and 3, in Fig. 4:

F = horizontal impact assumed to act at the center line of the shaft.

R = beam reactions plus any loads from the unit that are directly over the columns.

Replace F by the two forces, $F \times \frac{B}{W}$, and an equal force, F, acting at the top of the pedestal.

Compute the reactions, R_a and R_b, and the moments, M_4 and M_5.

Assume the column size and check for the fiber stress, f.

Let P = the maximum vertical load on column in question.

$$\text{For Bent (1)}, f = \frac{P}{A} + \frac{M_4}{S_{xx}} + \frac{M_1}{2S_{yy}}$$

$$\text{" " (2)}, f = \frac{P}{A} + \frac{M_4}{S_{xx}}$$

$$\text{" " (3)}, f = \frac{P}{A} + \frac{M_4}{S_{xx}} + \frac{M_3}{2S_{yy}}$$

(*d*) Design the bracket, diaphragm, or braces in the transverse bent to take the direct stress or moment, or both, according to the arrangement. Design the column bases, sub-foundation mat, and remaining elements.

4.—Provision for condenser and vacuum loading: If the condenser is connected to the turbine by an expansion joint and the weight is carried from the top of the pedestal, the weight of the condenser and water content, together with the vacuum load, must be included in computing the bending moment curves. If the condenser rests on beams framed into the columns, the bending moment diagrams for the columns shown must be modified. If the condenser is rigidly connected to the turbine, and supported on springs which are independent of the pedestal, add the load on the turbine joint allowed by the manufacturers to the other forces acting at the top of the pedestal.

5.—To care for axial forces, provide longitudinal brackets or bracing equivalent to the transverse bracing.

6.—Compute the weight of the complete pedestal and compare it with the typical weight curves.

A wide diversity exists as to proper impact allowances and working stresses, as will be noted in Table 3 which gives stresses recommended by various authorities for structural steel designs.

TABLE 3.

Reference consulted.	Stress, in pounds per square inch.	Impact assumption.	Stress, in pounds per square inch, reduced to condition of dead load without impact.
1.....	One-third the ordinary.	.....	5 333*
2.....	6 000	"To take care of all loading, including impact and vibration.".....	6 000
3.....	10 000	"For the total load, to which has been added 25% of the machine loading to cover impact.".....	8 000
4.....	16 000	100% impact for all steel work.....	8 000
5.....	Halve the allowable stress and double the calculated stress.	.....	4 000*

* Based on a standard working stress of 16 000 lb. per sq. in.

As stated previously, it is believed that this diversity of opinion is due to the failure to consider the horizontal forces to be sustained. In pedestal designs, the writer endeavors to make conservative assumptions as to all horizontal forces to be considered as well as vertical dead and impact loads, and to design for these forces, using working stresses that are standard for general structural designs. This practice has resulted in designs that have proved adequate in every respect. For steel members in bending, the deflection limit will result in the selection of sizes that will give stresses that are much lower than standard.

Probably in few cases of structural design must deflections be kept within such small limits, a common value for the maximum allowable deflection being 0.02 in. A continuous beam with fixed supports will deflect least under a given load, and the longitudinal girders should be designed to secure this condition.

These girders should be made continuous over the center columns. Absolute fixity of the end supports will not exist as some deflection at these points will occur, but the true bending moment curve may be approximated by averaging the ordinates of the "free end" curve and the "fixed end" curve (see the bending moment curves, *A* and *B*, Fig. 4). Columns 1 and 3, Fig. 4, must be designed for the bending moments, $\frac{M_1}{2}$ and $\frac{M_3}{2}$, respectively, in addition to their axial loads.

In the design of connections, gussets, and other details, assumptions should be conservative throughout, bearing in mind that vibrations should be restrained in all directions. Columns should be of ample section with relatively short unsupported lengths. In order to secure this condition, brackets should be made as deep as possible at the column heads, and it is sometimes advisable to provide transverse brackets at the column bases. Structural sections less than $\frac{3}{8}$ in. thick should be avoided. The use of long cross-bracings has been criticized as they tend to allow sway before they come into operation.

Good results have been obtained with designs in which the top of the pedestal is of open steel construction, with steel plates or gratings forming a floor at this level. In such designs, it is desirable to insulate the bearing surfaces carrying the plates or gratings with cork or felt, in order to eliminate vibrations of an annoying nature in the floor. A better design, however, is furnished by making the floor in the form of a thick reinforced concrete slab. This not only produces a vibration-absorbing mat at the most effective location, but also provides a good insurance against horizontal disturbance of the alignment of the unit. If a concrete floor is used, ample temperature steel should be provided, as the temperature range is greater than that under ordinary conditions, and cracks are likely to develop.

The close proximity of the live steam lead to the exhaust outlet for some types of turbo-generators often makes it difficult to insert a longitudinal beam of adequate size in this space. In the steel design shown in Fig. 1, the longitudinal girder at the turbine end has been located outside the live steam lead, thus avoiding this difficulty. This design would be improved by making the longitudinal girders continuous over the center columns. The concrete filling between the webs of the longitudinal girders shown is a common means often adopted to secure additional rigidity and mass.

Operating engineers have observed that sudden changes in the temperature of the basement may affect the even running of the turbo-generator, due to the unequal expansion of the steel columns of the pedestal. To avoid this difficulty, these members are sometimes encased in concrete to insulate them from sudden temperature changes. Some designers consider it advisable to encase all structural steel members in concrete for fire protection and to obtain additional mass. Fig. 5 illustrates the structural steel pedestal for one of the 30 000-kw. units at the L Street Station of The Edison Electric Illuminating Company of Boston, Mass., and Fig. 6 is a photograph of the same unit in operation, with the steel pedestal encased in concrete.

The sub-foundation below the basement floor level has been referred to as the footing or mat. In its design, the disastrous effects of settlement on the

alignment should constantly be borne in mind. In some designs, separate footings have been used for the individual columns and satisfactory results obtained, but it is better practice to design the footing as a single mat large enough to include all the columns. This mat should be of ample thickness to prevent local settlement, and it is often advisable to provide steel reinforcement to secure this end. Column footings are sometimes made to rest on steel grillages encased in the mat.

Certain details are common to practically all turbo-generator installations, and the designer of the pedestal should familiarize himself with them before starting his design. The important question of clearances for the condenser, auxiliaries, and their piping, should be borne in mind, and certified prints of the apparatus should be obtained at the start so that space for operation and dismantling may be assured. At this point, a turbo-generator manufacturer may be quoted as follows:

"The foundation must not extend inside the bed-plate facing where it may interfere with the ventilating ducts under the generator, and provision must be made for these ducts communicating with the cooling air. Provision must also be made for bringing out leads under the bed-plate and through the foundations if necessary.

"Foundations must be designed with the idea constantly in mind of the complete installation of the unit and possible future repairs. Provisions must be made for the removal and replacement of parts such as oil cooler, pump, steam chest, throttle and inlet, exhaust and bleeder piping connections, and supports, and also exhaust connections, expansion joints, and atmospheric outlet piping. This means that the question of interference must be carefully gone into in connection with all parts approaching the plane of the lower surface of the bed-plate and extending below the same. The positions of the centers of loading and magnitudes and the location of loads are given on the official outline drawing sent to purchaser."

The manufacturers' plans will give dead and vacuum loads, clearances necessary for piping and air-ducts, maximum allowable deflections, anchor-bolt location, if bolts are to be used, and other information of a general nature. The instruction book for the type of unit under consideration will also be found to contain many valuable suggestions. Prints of the footing and pedestal should be sent to the manufacturers as soon as practicable for their suggestions, so that their final comments may be obtained before letting the contract for the work.

If a surface condenser is to be used, a trolley beam should be provided to facilitate the removal of the condenser head. If the generator is to be bolted down, provision should be made for removing the bolts from below in order to avoid raising the unit for dismantling. To avoid the transmission of vibrations, it is generally considered advisable to isolate the pedestal and footing from the remainder of the power plant.

Curves are shown in Fig. 2 giving the estimated costs per kilowatt of pedestals (excluding the sub-foundation or mat) for several existing installations, and for certain typical designs suggested by manufacturers of turbo-generators. Present-day prices have been used, and percentage charges, such as profit, overhead, bond, and liability insurance, are included. It will be noted that, except for the smaller sizes, there is no great difference in cost between



FIG. 5.—STEEL SKELETON FOR PEDESTAL FOR 30 000-KW. TURBO-GENERATOR.

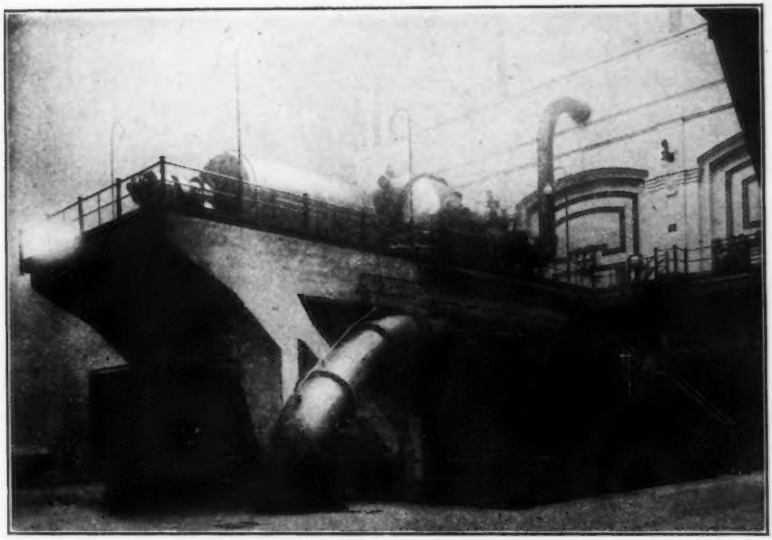


FIG. 6.—COMPLETED PEDESTAL FOR 30 000-KW. TURBO-GENERATOR.





the concrete pedestal and the steel pedestal. For ordinary designs, the cost of the pedestal should not exceed about 5% that of the turbo-generator, and, in general, any saving of one particular type of pedestal over another will not be more than about 1% of the cost of the unit which it supports. Considering the cost of the turbo-generator, and the important part in its satisfactory operation that its supporting pedestal plays as to freedom from vibrations and accessibility, it is believed that the latter's adaptability to the operation of the unit should be the only consideration in deciding which type of pedestal to adopt. Each type has its field. For the smaller units, it is believed that more concrete pedestals are in use, whereas, the reverse is probably true of the larger units. The composite type, combining the advantages of both concrete and structural steel, is coming to be more and more favored for all sizes of turbo-generator units.

DISCUSSION

W. E. BELCHER,* M. AM. SOC. C. E.—The development of the large steam turbine generator unit marks a rapid advance in the science of Mechanical Engineering, which has made necessary the special structure discussed by the author. It is to-day an appropriate topic for consideration, although ten years ago it was in its infancy, and ten years hence may have lost all practical interest.

At the World's Fair in 1904, one's attention was attracted by a loud shrieking noise which came from a steam turbine forming part of the exhibit of the Westinghouse Company. This exhibit was perhaps the introduction of the steam turbine to American engineers. For a number of years thereafter, the adoption of the turbine was very slow, the usual form being the vertical unit with capacities of 2 000 to 5 000 kw. These units were mounted on circular or octagonal concrete piers that were hollow in the center for access to the step bearings. This form was virtually confined to the impulse type of blades, and did not lend itself to expansion in the capacity of the unit.

About 1914, the manufacture of the vertical turbo-generators was discontinued, and during the last ten years, the horizontal type has rapidly been replacing all other forms of electric power generators in central station practice.

It is to be regretted that the author's paper did not show a greater range of actual design data applicable to units larger than 15 000 kw., and, in order to be of the greatest usefulness, the discussion of the paper should present examples of current practice with large units.

In the speaker's experience, single units with capacities as large as 35 000 kw. have been mounted on frames of the type described in the paper. A unit of 60 000 kw. has been installed at Cheswick, Pa., in which case, however, there were three separate turbines, one high pressure and two low pressure, each with a separate support. In designing these supports, it is the speaker's practice to use the following maximum unit stresses in structural steel members:

Compression on columns.....	$8\,000 - 45\,\frac{L}{R}$ lb. per sq. in.
Bending on extreme fiber of net section	8 000 lb. " " "
Rivet values, shop-driven, in shear	6 000 lb. " " "
Rivet values, shop-driven, in bearing	12 000 lb. " " "
Field driven rivets.....	four-fifths of the values given

Good judgment is more important, however, than fixed unit stresses, and the quantity of steel and concrete used should be as great as is reasonably

* Structural Engr., Dwight P. Robinson & Co., New York City.

justified rather than as small as possible. In erecting the machine, the top of the rough concrete slab and the steel frame are left about 1½ in. below the base casting, which is grouted in solid after all work has been aligned and leveled. Although the entire weight of the machine is in place, and carried on shims above the rough concrete slab, the framework of the support will take whatever deflection the load may cause. With this form of construction, therefore, the entire deflection is taken care of by the grouting, so that the speaker does not consider deflection to be the criterion.

The speaker believes that provision should be made for the full weight of the condenser where the connection between condenser and exhaust nozzle is bolted solidly, even if springs are placed under the condenser.

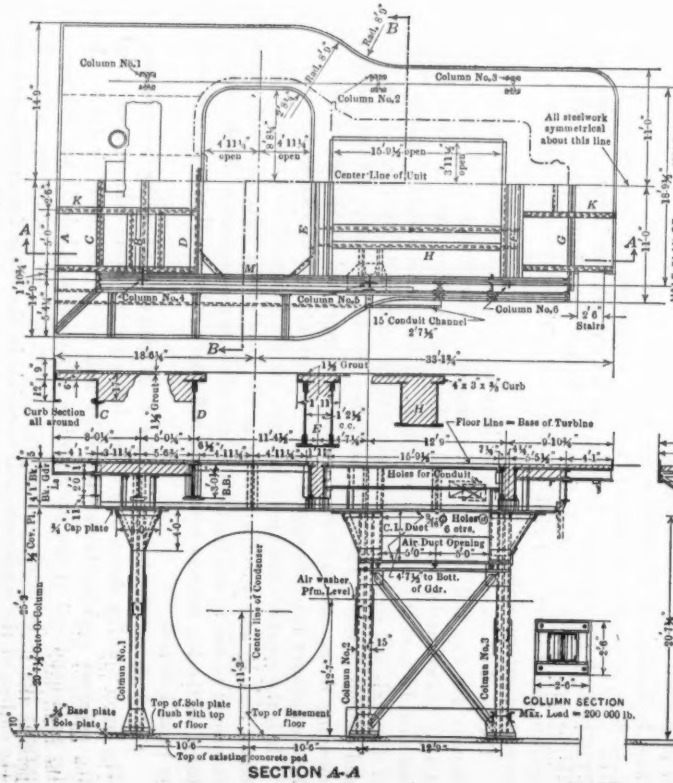
Co-operation between the mechanical and structural engineers is essential in the early stages of the design, and a slight change in the mechanical layout of the auxiliary equipment may permit the placing of a diagonal or the deepening of a girder, thus improving considerably the structural design.

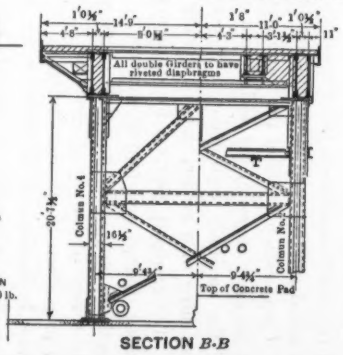
The cramped quarters for equipment beneath the unit, including the condenser, pumps and piping, generator, air inlet and discharge, electrical connections, etc., make a design of reinforced concrete especially difficult. If the severe limitation of space as it develops makes one of the members out of proportion to the remainder, it is likely to cause trouble. As a result of these limitations, steel supports are generally more satisfactory. The platform floor is usually of concrete and the supporting girders are filled with concrete, both for structural bracing and for the mass resistance to vibration. This is well shown in Fig. 7, the spaces between the double girder webs and between the stringers and girders being filled with concrete, in addition to the concrete platform slab.

Every effort should be made to have the main longitudinal girders continuous for the full length of the support. The longitudinal bracing is usually placed under the electrical end of the machine, but transverse bracing must be used at each transverse bent. This bracing should be in the form of diagonals and struts, as far as possible, rather than in the form of brackets and column flexure. The entire structure should be separated from the surrounding floors, and platforms or walkways supported by slender hangers or light cantilever brackets should be avoided on account of vibration.

It is the speaker's practice to assume, for the design of the bracing, a horizontal load in any direction acting at the level of the main platform, equal in amount to one-fourth the total weight of the turbo-generator. Such a design based on the low stresses mentioned will give a substantial system of bracing in each direction.

Fig. 8 shows the foundation for a 20 000-kw. unit under construction. The larger opening shown on the top slab is for the steam exhaust nozzle, and the narrower opening is for the inlet and outlet generator air-ducts. At the right-hand end of the plate girders, a rectangular hole in the web-plate, permits the main generator cables to enter the space under the generator. The outline of the condenser shell is visible in the lower foreground, and the platform for the air washer and intake appears at the right. Fig. 9 shows





DATA ON GIRDERS

Mark	Shear in pounds per square inch	Moment in foot pounds	Web	Flanges
A	6000		1 12" Channel 20.7 lb.	Curb Pl. 9"x ⅜" Curb L 4"x 3"x ⅜"
B	35 000	173 000	1 24" 105 lb. I	
C	28 000	163 000	1 24" 105 lb. I	
D	75 000	338 000	1 36" x ½ Pl.	4 Ls 6"x 6"x ⅝" 1 L 4"x 3"x ⅜"
E	120 000	506 000	2 36" x ⅜ Pls.	4 Ls 6"x 4"x ⅝" Top 2 Ls 4"x 3"x ⅜"
				4 Ls 6"x 4"x ⅜ Bott. 1 Pl. 23"x ⅜
F	100 000	479 000	Same as E	except only 1 L 4"x 3"x ⅜"
G	38 000	143 000	Same as C	
H	55 000	235 000	2 24" 80 lb. Is	1 L 4"x 3"x ⅜" Pl. 27"x ⅜"
M	168 000	829 000	2 Pls. 48"x 5/16	Top 1 L 4"x 3"x ⅜" 4 Ls 6"x 4"x ⅜"
				Bott. 4 Ls 6"x 4"x ⅜" Pl. 21"x ½"
L			Same as M except omit 2 Ls Top and 2 Ls Bottom	
K			12 1 40 lb.	

Columns Web 15"x ½"
4 Ls 6"x 6"x ⅜"
2 15" 33 lb. I.

20 000 K.W. SUPPORT
NEW ORLEANS RAILWAY & LIGHT COMPANY

FIG. 7.

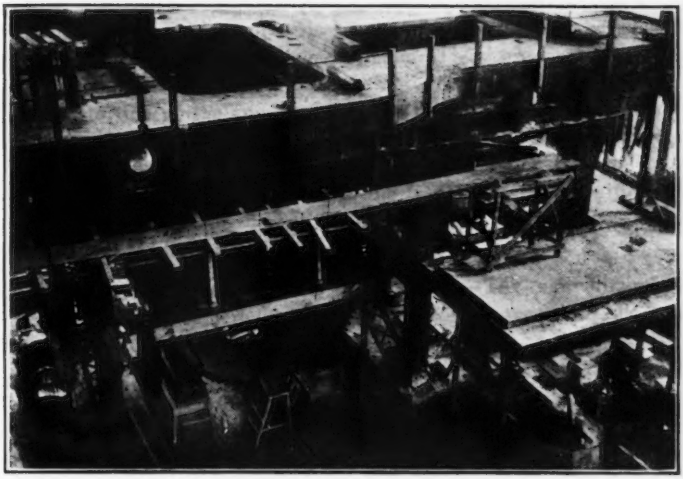


FIG. 8.—FOUNDATION FOR 20 000-KW. TURBINE, FOR NEW ORLEANS RAILWAY AND LIGHT COMPANY, NEW ORLEANS, LA.

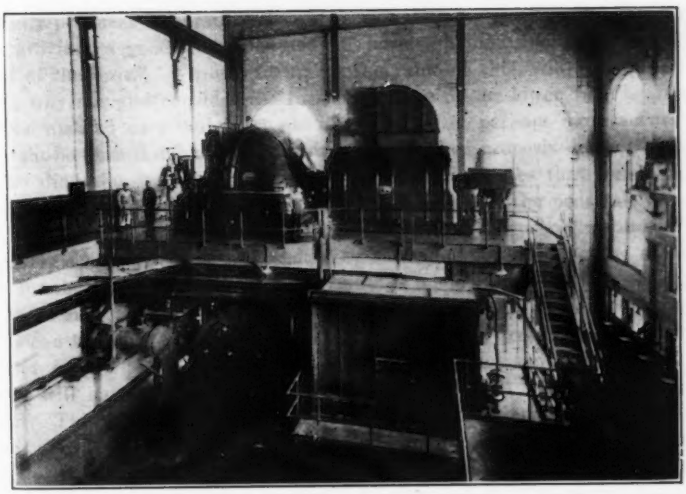


FIG. 9.—VIEW OF 20 000-KW. TURBINE IN OPERATION, IN MARKET STREET STATION OF NEW ORLEANS RAILWAY AND LIGHT COMPANY.



the general view, with the machine in operation, in the Market Street Station, of the New Orleans Railway and Light Company, New Orleans, La.

Even if a structurally perfect design has been secured, experience has shown that local conditions may cause trouble. There are cases recorded in which one column stood in direct sunlight while others were in shadow, thus causing a distortion of the frame and the destruction of the generator. In another case, distortion was caused by a cold draft at one end of the frame and considerable expense was entailed until the cause of the trouble was found. It is essential, therefore, to keep the frame at a fairly uniform temperature, and, with this in view, the steel members are sometimes encased in concrete, as noted by the author.

AUSTIN H. REEVES,* ASSOC. M. AM. SOC. C. E.—The main difference of opinion between Mr. Cameron and the speaker is in regard to the use of the "footing" or "mat". The speaker believes:

(*a*) That "separate footings" should never be "used for the individual columns";

(*b*) That it is always advisable to provide steel reinforcement in the footing;

(*c*) That steel columns should always be suitably riveted to a proper grillage in the mat or be of proper form at the base to make full use of the concrete mat. (Anchor-bolts in the ordinary sense of the word are not sufficient.)

On page 331, the author states that "a better design, however, is furnished by making the floor in the form of a thick reinforced concrete slab. This not only produces a vibration-absorbing mat at the most efficient location, but also provides a good insurance against horizontal disturbance of the alignment of the unit." While recognizing the value of this "vibration-absorbing mat", why not also consider the full, proper use of the lower "mat". It may also be utilized as a "vibration-absorbing mat", or, perhaps more accurately, it might be termed a "vibration-lessening mat" when properly designed. None of the diagrams, tables, or curves in the paper indicates that consideration extended below the top of the "mat". In speaking of the pedestals on page 326, the author states, "* * * it is believed that all turbo-generator manufacturers concur in the desire that they be made very rigid." The speaker's plea for the full scientific use of the "mat" is made because such use will give the most rigid pedestal. By proper design of the "mat", the axes of the columns can be kept vertical just above the mat, which is equivalent to reducing H and L in Fig. 3, and with the same pedestal and supported weight, the frame will have a shorter natural period of vibration, which is a desirable result. This helps in a manner similar to that of lowering of the center of gravity "by hanging the condenser from the top of the pedestal". The author has stated that "the more rigid a pedestal is made, the shorter its natural period will be, and its members should be arranged so as to secure this end." The speaker claims the "mat" is a very important "member" of the most scientifically designed foundation for the support of a turbo-generator.

* Newark, N. J.

F. A. Ericson,* M. Am. Soc. C. E.—During the speaker's connection with the New York Edison Company, and for the last few years with Thomas E. Murray, Incorporated, a variety of turbine foundations have been designed, the capacity of the machines ranging from 7 500 to 50 000 kw., in all, 558 000 kw.

The steel design has been adopted for the following reasons:

1.—It allows more room for piping and auxiliaries and, therefore, enables a greater capacity of a given space or building to be obtained, thereby reducing the cost per kilowatt.

2.—It is believed that a better, more serviceable, and more durable foundation can be built of steel than of either plain or reinforced concrete.

3.—In some cases, the space is so limited as to exclude all possibility of using any other type.

The design adopted by the speaker's Company consists of two main longitudinal girders, connected by a system of deep and stiff cross-girders and auxiliary longitudinal girders where they were necessary, in order to provide sufficient support for the bed-plate of the turbo-generator. The columns are placed directly under these longitudinal girders, with bracing inserted wherever practicable both longitudinally and cross-wise. All the girders are provided with full-depth, heavy angle connections to each other, and all connections to columns and bracing are stiff and substantial. Wherever the girders are in clusters, they are held together with bolts and gas-pipe separators, and the space between them is filled with concrete. This increases the lateral stiffness of the foundation to a great degree and also adds considerable weight, each of which tends to reduce vibrations. In designing the girders, a very low fiber stress of 5 000 to 6 000 lb. per sq. in. is used in order to minimize vibrations, and the allowable shear is reduced in proportion. Although this low fiber stress is used, nevertheless it is found necessary to increase the material in most of the girders, especially on the larger units, otherwise, the deflections would become excessive. A force proportional to the turbine load is assumed as acting in horizontal directions and is used in designing columns and bracing, but the bracing is always made extra heavy and direct acting wherever possible. All the details are designed with the idea of making a monolithic and continuous structure.

In order to avoid the possibility of unequal settlement of columns, the turbine foundation is set on a thick mat of continuous reinforced concrete, which is usually a part of the foundation for the operating room.

James H. Richardson,† Assoc. M. Am. Soc. C. E.—The speaker has had more or less to do with the designing of turbo-generator supports and wishes to present a friendly plea for the structural man as against the machine designer. The latter usually precedes the structural designer in planning power-plant units, arranging the condensers, ducts, and ramifications of pipes, large and small, often with too little regard for the structural parts of the unit.

* Structural Engr., Thomas E. Murray, Inc., New York City.

† New York City.

If the designer of the supports could rely on the strength of curved columns and sinuous girders, all might be well. As this is not the case, "let the mechanical designer allow plenty of space for the most favorable arrangement of the structural members necessary for rigidly supporting the turbo-generator unit."

F. M. Beer,* Assoc. M. Am. Soc. C. E.—Although not thoroughly conversant with turbo-generator practice, this paper brings to mind two installations, one of which is of 1 500-kv-a. and the other of 3 000-kv-a. rating, that are in use in an electric light and power plant in New York City. The supports of these units seem quite well to overcome the objectional vibrations set up while the machines are operating.

Two concrete walls of uniform thickness, parallel to the axes of the units, support transverse steel beams that carry the units. This arrangement appears best to meet the requirements of this class of equipment, as it provides the mass necessary to absorb vibration, where it is most effective and may be adapted to any size of unit. Access through these walls may be made wherever convenient and rigidity maintained even if the walls are chased or pierced to accommodate auxiliaries or piping.

George A. Orrok,* M. Am. Soc. C. E.—Since 1905, the speaker has been interested in the design of about forty steel foundations of various sizes. Most of the foundations built by the speaker's Company are of the table-top type. Two other types that are used, might be called the trestle type, in which the girders are athwartships of the turbine, and the girder type, in which two longitudinal girders not tied together rigidly, are used. In the last two types, the legs are bolted down, or an attempt is made to anchor them at the foot, which, in the table-top type, is not so necessary.

The total weight of turbine foundations should usually be adjusted to the weight of the turbine and condenser. Sufficient weight is added by concrete which is placed between the girders.

In the design of the table-top type, the deflection is kept less than about 0.006 in. Sometimes, a deflection of as much as 0.0013 in. in the frame of the machine will not cause serious trouble in its operation. Recently, a machine of 25 000 kw. capacity was placed about 1 in. above the steel cross-girder, at the middle bearing. Through some misadventure, the grouting worked loose, or disintegrated, and the steel wedges used had backed out a little, allowing the middle bearing of the turbine to drop about 0.0013 in. The turbine ran for a month or more in this condition until it could be dismantled and wedges again driven.

Whether the shaft must be level has also been a question. The speaker has noted cases where one end of the shaft was as much as 1 in. lower than the other. In these cases, a thrust-bearing was used, and the turbine worked satisfactorily.

There are four methods of supporting a condenser. Formerly, a concrete pedestal was used, an expansion joint being placed between the condenser and

* Cons. Engr., New York City.

the turbine. The concrete walls made repairs under the condenser difficult. About 1911, however, the practice was begun of hanging the condenser from the main girders, supporting the turbine on top of such girders, omitting the large expansion joint, and using expansion joints on the small water and air pipes. This made a good arrangement which is being used to a great extent at present.

In another case, spring beams were used between the columns supporting the foundation. The beams were designed so that they would spring sufficiently to allow the condenser to take its proper place. This construction has been used for nine years, without any trouble. The spring support was introduced about the same time and has been successful.

The Westinghouse Company is bringing out a new type of steel foundation. This foundation is in two parts, which allows the condenser to be placed at right angles to the turbine, with the square, or oblong, foundation supporting the generator on one side, and the turbine on the other side, of the condenser. In this case, the condenser shell is made a part of the exhaust nozzle of the turbine itself, no large expansion joints are required, but smaller expansion joints are placed on the small pipe as before.

R. VON FABRICE,* ESQ.—The author has entered a field of Civil Engineering most intimately connected with the Mechanical and Electrical Professions. As a matter of fact, the subject treats of the execution of the design of a structure, and may mean either success or failure in the proper functioning of the prime mover of the modern power station. Therefore, it is recommended that the American Society of Civil Engineers should try to formulate specific recommendations covering the various phases of the design of structural supports for turbo-generators, which could be used as a basis in executing the designs of these important structures.

Structural steel supports, as foundations for prime movers, are of comparatively recent origin, the earlier installations having caused considerable difficulty, chiefly due to lack of knowledge pertaining to the specific characteristics of the equipment to be supported. This lack of proper information was not confined solely to the civil engineer, but was shared equally by mechanical and electrical engineers. It is not an infrequent occurrence, at present, for builders to be inconsistent in their instructions and data concerning the deflection allowable, the distribution of loading, and the rigidity of the component parts of the equipment.

Most of the manufacturers do not depend on the base-plates of their equipment for equal distribution of loading, and, consequently, the base-plate serves only for centering the component parts of the unit. This, however, is only true for those turbo-generators which have a continuous base-plate. Some of the builders of larger turbo-generators do not even provide continuous base-plates for their units, and thus depend solely on the foundations or supports to hold the various parts in alignment, supplying an additional difficulty to be overcome by the civil engineer, and, incidentally, also greatly increasing the cost of the structural supports.

* Designing Engr., Public Service Production Co., Newark, N. J.

The cost of structural supports for turbo-generators having well-designed, continuous, rigid and adequate bases, is approximately 25% less than the cost for the other and more commercial types.

The main difficulties encountered in the design of the structural supports are the space limitations on account of auxiliary equipment, the limitations of allowable deflection in the structure, and inadequate provisions for the supporting or bearing surfaces on parts of the turbo-generators themselves.

The three types of pedestals mentioned by Mr. Cameron, namely, reinforced concrete, structural steel, and the combination structural steel and concrete, or composite type, are not adaptable to all kinds of installations, and the proper selection of the type to be used for any specific case is dependent on the auxiliary equipment, type of unit, space conditions, and arrangement of equipment.

The structural steel type has the greatest advantages, as it provides the most accessible arrangement for the auxiliaries and piping, and, further, possesses the greatest positive mathematical determination of design.

The composite type consists of a structural steel pedestal entirely encased with heavy concrete. The main practical advantage is that the encasement of concrete provides volume or mass, and incidentally a protection to the structural steel against temperature changes and consequent distortions of the structural frame. This last item, however, can also be obtained by a 2 or 3-in. thickness of concrete over the structural steel members, and this protection is of greatest value for the columns. The structural value of the heavy concrete encasement is uncertain, and it is advisable to ignore the value of the concrete in the calculations for strength and rigidity. Therefore, it would seem advisable to encase the steel only against temperature distortion, and design the steelwork as a purely structural support, of adequate strength for deflection and vibration.

Sudden and local temperature changes in the turbine room basement will distort the structural type of pedestal sufficiently to cause serious disturbance in the turbo-generator itself. Therefore, some engineers believe it advisable to protect the columns against such temperature changes. In any event, such protection can be obtained at a comparatively small additional cost.

The reinforced concrete pedestal is becoming less popular in modern installations, and is unsuitable for a compact and accessible arrangement of auxiliaries. Furthermore, it is impossible, in most instances, to obtain the structural strength in cross-girders, as required by the limitations of deflections imposed by the manufacturers and, also, by the inadequate bearing surfaces provided on the equipment itself at the points of most critical concentration of loading. However, this is not the case for units when continuous, heavy and properly designed base-plates are provided by the manufacturers.

Vibration in the pedestal can be reduced to the minimum in any type of pedestal, and is dependent on the proper methods of bracing, design of connections, proper detailing of connections, and primarily on the proper balancing and adjusting of the unit itself.

To provide properly against vibration in pedestals of the structural type, it is imperative that bracing in all directions be supplied, and that the

columns be secured to the grillages and girders with heavy brackets and knee-braces.

It is advisable to cross-tie the grillages so as to have at least one beam of the grillage continuous under the entire set of supporting columns in both directions, and to provide heavy brackets riveted to the column and grillage. This will stiffen the entire structure and guard against looseness in the connections between the columns and grillage. Further, all tiers of the grillage should be riveted together, and the column base should be bolted or riveted securely to the grillage.

All tension and compression members should be fabricated to provide initial tension or compression, and in practice it has been found advisable to fabricate tension members $\frac{1}{8}$ in. short in 10 ft. and compression members $\frac{1}{8}$ in. long in 10 ft. This will eliminate all possibility of slackness in the joints. It is also advisable to increase connections for all abutting members by at least 50% of the theoretical connection value, in order to provide rigidity at the joint itself and eliminate loosening of connections later. This further tends to throw the oscillations or vibrations into the members themselves, rather than into the joints or connections.

All girders composing the platform of the pedestal supporting the turbo-generator should be designed for deflection only, and the summation of all deflections of all abutting or connecting girders should result in a uniform deflection under all pedestals or bearings of the turbo-generator along its longitudinal center line.

The top of the steelwork, for a structural support, should be arranged so that at least 4 in. of concrete and grout are possible between the base of the turbo-generator and the steelwork to insure against pulverization of the grout after a period of operation.

The space between the girders of the platform should be filled with concrete in order to provide a massive mat to absorb the pounding of the unit and add to the rigidity of the joints. All parallel supporting girders under any supports of the turbo-generator should be tied together by heavy plate and angle diaphragms. Pipe and rod or standard cast-iron separators are not reliable for this purpose.

Adequate provisions should be made for foundation bolts holding the turbo-generator to the pedestal, and in cases where supporting girders come in line with holding-down bolts, a specially shaped billet, tapped for the holding-down bolt, is to be securely riveted to the top of the girder.

In the design of structural steel pedestals it is not safe to base the design on any fixed unit fiber stress as a limiting feature governing the size of the girders. Each girder should be designed purely for the proper required deflection for the load to be supported. The designer must make proper allowances for impact and distribution of loadings or weight of the unit on the various girders of the platform.

For the design of girders, three times the actual net weight of the unit has been found to be ample to provide for impact and dead weight of the structure itself. For the design of columns and grillages, three times the actual weight of the unit, together with the weight of the platform, should

be used. Grillages spanning a canal, or not being supported uniformly, should also be designed for deflection, and this deflection should be considered in the design of the platform and entire structure.

Continuous girders parallel with the longitudinal axis of the turbo-generator should be designed as such, and all positive and negative moments should be considered. Ample stiffness should be provided at all cross-connections and over columns. Turbo-generators which have a substantial, well designed, and self-supporting base-plate do not require continuous side girders, and, consequently, a less expensive pedestal can be surely designed, which can also consist of two separate structures, if so desired, supporting separate parts of the unit.

The entire structure should be supported on a continuous heavy concrete mat of adequate thickness to guard against shear and to avoid local settlement under any individual column. Steel reinforcement is required when the volume of concrete available is of limited thickness.

When setting the turbo-generator on the pedestal, it should be wedged securely to the proper alignment with steel wedges. The unit should then be grouted in place with a rich cement grout which should be poured so as to fill practically the entire hollow part of the base-plate, as this will provide for shrinkage as the grout dries and hardens under the unit. The steel wedges should not be removed, but cut off, when they extend into openings required for air ducts, piping, or other auxiliary parts.

When installations are made, care should be taken to avoid any rigid connections between the structural support of adjacent units, building wall, and building steelwork. This is primarily important in the direction at right angles with the longitudinal axis of the unit. All galleries around the turbo-generator, and secured to the pedestal structure, should have all grating and cover-plates (if any are used) bolted securely to the steelwork, in order to avoid unnecessary disturbances. If concrete floor-slabs are supported on these galleries, they should be haunched on the lower flanges of the supporting beams, to insure against loosening from the supporting steel and consequent disturbances.

The method of supporting the condensers depends largely on the size of the unit and the design of the condenser and turbine.

When an expansion joint of adequate design is used, the condenser can be supported either on pedestals from the basement floor, on steelwork connecting to the columns of the pedestal, or from hangers from the supporting platform itself. The latter method is not to be recommended, as it adds undue loading on the girders supporting the turbo-generator, and consequently increases greatly the cost of these members, on account of the small allowable deflections in these girders.

For large installations, the method of supporting the condensers on properly designed and adjustable spring supports is the most satisfactory, especially if there are provisions for testing the actual distribution of the condenser weight by suitable hydraulic jacking devices. With this arrangement, accurate distribution of loading due to the condenser is possible, and the proper amount of dead weight required by the builders on the casing

of the turbine due to the condenser is insured. Also, blocking devices should be provided to furnish proper means for supporting the condenser when filled with water for testing purposes, in order to guard against undue strains on the turbine casing under these conditions.

For new turbine-room designs, ample head-room should be allowed, in order to provide space for all auxiliaries, so as to avoid the necessity of providing pits under the equipment for condensate pumps and other auxiliaries. The following figures will provide sufficient head-room between the basement floor and top of the turbine foundation for the various units:

For	units	from	30 000	kv-a.	to	45 000	kv-a.,	or	larger....	35	ft.
"	"	"	20 000	"	"	30 000	"	"	"	30	"
"	"	"	12 000	"	"	20 000	"	"	"	25	"
"	"	"	5 000	"	"	12 000	"	"	"	20	"

D. A. ALLEE,* M. AM. SOC. C. E. (by letter).—This paper enters a field for speculation and discussion not to be lightly dismissed. Without in any way discouraging such speculation, the writer would like to present the situation as it is found to-day.

Three types of foundations have been mentioned, namely, reinforced concrete, structural steel, and a combination of structural steel and reinforced concrete, designated "composite". When designs are carefully worked out, giving due consideration to all conditions, it is believed that all types have their fields, and that any type may fairly be expected to give satisfaction; but therein lies the trouble. Which designs in practice turn out to be best? Good foundations rather than good theory are sought.

From long experience in this field, it is found that more steel structures are insufficient in their conceptions and details than is the case with concrete, as applied to turbo-generator foundations. For example, there is a tendency to use a top construction which, in itself, has no ability to maintain alignment in a vertical plane. There is also a tendency to support and grout machine bases on yielding surfaces that, therefore, allow the grout to break. An example of this is a girder with outstanding flange angles. There is, likewise, a tendency to use bracing and stiffening construction, which in themselves are quite effective, but which in practice cannot be consistently carried out, for example, heavy brackets connecting girders to columns giving the appearance of great stiffness. If, after adopting such a type of construction, it is impossible to use such brackets on one side of the foundation because the girder cannot be placed over the column, or if it becomes necessary to move a cross-girder to a position not directly over the column, as is the case with practically all turbines of a certain design, the whole principle of the design has been violated.

In general, it will be found that steel foundations are insufficient of themselves and must be tied to the floor on which they depend for support against swaying and for alignment in a vertical plane.

The composite (framed steel and concrete) design always presents difficult problems, where it is attempted to connect steelwork to concrete or to support steel on concrete. This is particularly true with large machines where loads are excessive. If steel girders are framed to steel columns and simply covered

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with concrete, protection against total failure of the structure is obtained, but it is still necessary to reinforce the concrete. The function that this concrete cover performs is problematical. If the details are such (they seldom are) that dependence may be placed on the concrete, then the steel core is superfluous. In general, there is no great advantage to be gained in this construction over that of reinforced concrete.

Designs of reinforced concrete and those of all steel should be compared on a common basis. For example, referring to Fig. 1, it is quite apparent that the steel foundation is not suitable for the conditions shown in the concrete design. It is further apparent that the concrete design may be made much lighter if desired, causing the weight to be less and the clearances to be greater, but at a sacrifice of simplicity, and yet have the design the equivalent of the steel foundation with which it is being compared.

It should be kept in mind that such structures are designed by various grades of talent and, therefore, a design which would prove safe and effective, if made by a competent and careful engineer, might and probably would be unsatisfactory if developed by one less competent. Of the multitude of designs which the writer's Company examines, practically none is above criticism in the essentials of good design. It is found that poorly conceived designs in steel construction are so common that the engineers of the Company have been very reticent about encouraging their use.

EDWARD H. CAMERON,* ASSOC. M. AM. SOC. C. E. (by letter).—Two points brought out by the discussion of this paper are worthy of especial emphasis. The first is the desirability of close co-operation between electrical, mechanical, and civil engineers who are working together in power-plant design. A better foundation can be secured if the civil engineer on his part clearly understands the functions of the machine and its component auxiliaries. The mechanical and electrical engineers on their part must recognize the absolute necessity of providing space for the more important bracing elements and should be willing to modify their layout somewhat to make possible the insertion of such members. The second point considered worthy of emphasis is the importance of experience and good judgment on the part of the designer as against an attempt to adhere strictly to fixed unit stresses.

The writer believes that the present custom of fixing a limit to the deflection of members in bending constitutes a very desirable criterion of the rigidity of the pedestal. It has been stated that the entire deflection is taken care of by the grouting. The writer cannot agree on this point. Before grouting, the unit is brought to a level by steel wedges. If light girders are used, it is apparent that the relative stiffness of the girder and of the bed-plate will determine whether the driving of the wedge will tend to raise the bed-plate, or merely drive down the girder and allow the bed-plate to deflect with it. Consequently, the girders should be made sufficiently stiff that any load which they support, other than their own weight, will not cause a deflection greater than the limits ordinarily set.

Mr. Reeves is a little more positive than the writer regarding the undesirability of separate footings for the individual columns. The writer believes

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with Mr. Reeves that the continuous mat should be used wherever possible; however, it is not always practical to secure this condition, and several very good designs have been noted in which the columns rest on individual footings. Both Mr. Reeves and Mr. von Fabrice have emphasized a very important point relative to the delivery and distribution of the column loads and moments into the mat. Following out the analysis given in Fig. 4, Moments M_1, M_3, and M_5, should be cared for at the column bases.

Mr. Beers mentions a type of pedestal consisting of two longitudinal concrete walls carrying transverse steel beams which, in turn, support the turbo-generator. The writer has noted several similar installations where a jet condenser was used. This design should be quite simple to build for small units as the deflections would be limited to the comparatively short transverse beams.

Several of the discussors have raised the question as to the desirability of placing reliance on the structural strength of the concrete encasement of a steel pedestal. It is very difficult and often impractical to make a theoretical analysis of the proportions of the load that would be supported by the steel member and the concrete encasement, respectively. The writer believes that the recommendation of Mr. von Fabrice that it is advisable to ignore the structural value of the heavy concrete encasement, is justified and should form a part of any conservative specification for this type of work. The concrete encasement would then be considered only for additional mass and temperature insulation.

Mr. von Fabrice quotes certain standards of shop fabrication which have been found advisable, such as making tension members $\frac{1}{8}$ in. short in 10 ft. and compression members $\frac{1}{8}$ in. long in 10 ft. There are no structures made up of the ordinary rolled steel sections in which especial refinements of common bridge shop practice are more justified.

The writer is in hearty accord with Mr. Allee in that "it is good foundations * * * that are sought". A substantial completed structure is the object of the designer in all civil engineering fields, and there is no question but that good theory based on successful practical application is the best.

In presenting Fig. 1, comparing a structural steel foundation as recommended by one manufacturer with a reinforced concrete type given by another builder, the writer did not intend to imply that the machines would be interchangeable on the foundations, or that they were precisely equivalent. The writer believes, however, that the examples selected are largely comparable and present a striking illustration of the distinct advantage of the structural steel pedestal in regard to space as contrasted with the concrete type.

In the discussion of this paper, there is a very evident preference shown by most of the discussors for the structural steel foundation, perhaps modified by the application of a concrete encasement and slab. If the manufacturers are finding steel designs to be as poorly conceived as intimated by Mr. Allee, it is high time that steps are taken to correct this condition. The writer believes that consideration should be given to the suggestion of Mr. von Fabrice that standards for the design of structural supports for turbo-generators be formulated by the Society.

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ANALYSIS OF COST OF FREIGHT SERVICE, GRAND TRUNK RAILWAY COMPANY OF CANADA

By J. P. Newell,* M. Am. Soc. C. E.

With Discussion by Messrs. J. G. Sullivan and J. P. Newell.

Synopsis

A part of the evidence presented to the Board of Arbitration appointed to determine the price to be paid by the Canadian Government for the preference and common stock of the Grand Trunk Railway Company was a study of the cost of freight service. For many years, the Grand Trunk Railway Company has sought traffic from Chicago, Ill., *via* Montreal, Ont., Canada, to New England ports, and even to New York City, in competition with the shorter American lines. This business is necessarily handled at high speed and low rates per ton-mile. The immediate purpose of the investigation was to determine whether this traffic is carried at a loss, but the methods used are here presented as applicable to the operations of any railroad, their specific application to the Grand Trunk Railway being used only for the purpose of illustration.

The general plan is one of analysis. The entire cost is assigned to various operations and, therefore, all unforeseeable happenings and accidents for the year are taken into account. The operations are divided into units, and the average cost of each is determined. In applying the results to specific cases, allowance is made for departures from average conditions. The year 1913 was selected as the latest one free from the influences of the World War.

Classification of Costs.—The cost of railroad transportation is composed of two principal elements: "Operating Expenses" and "Interest", including in the latter all charges of the same nature as interest. The Operating Expenses are divisible into Direct Costs, or those dependent on traffic, and Indirect Costs, or those not affected by traffic. Each class is also divided between the cost of Freight and Passenger Service. The cost of passenger

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service has not been considered by the writer, except as was necessary in separating it from that of freight service.

The direct cost of freight service is divisible into two classes:

(1).—Terminal cost, or the expense of originating and delivering traffic independent of the distance moved. This cost has two subdivisions: (*a*) Carload; and (*b*) Less than Carload.

(2).—Road cost, or the expense of moving freight between the points of origination and delivery. This cost is divided between: (*a*) Distance, or the costs which are dependent on the distance moved; and (*b*) Resistance, or the costs which are dependent on the resistance overcome on account of friction and gradients.

Interest.—The interest charge is commonly regarded as "fixed", but, in reality, a considerable part is variable, increasing as traffic increases. This part is segregated and classified under the same heads as operating expenses.

Units of Service.—The next step is to determine the number of unit operations performed during the year, under each of the classes into which the costs have been divided.

The unit of Terminal Carload operations is a single terminal handling of a car, either of origination or delivery. The unit of Less than Carload operations is one terminal handling of a net ton.

The units of operation in the Distance factor are the train-mile, the car-mile, and the gross ton-mile.

The unit of Resistance is the resistance equivalent to that of lifting 1 ton 1 ft., or of 1 ft-ton.

In calculating the cost of any specific freight movement, the carload is taken as the unit. The service units connected with the movement are calculated, and the respective costs applied to obtain the total, which is then divided by the number of tons in the car, to ascertain the cost per ton, or ton-mile. The indirect cost must be borne by the existing traffic, whether it is large or small, and may be apportioned according to any desired unit of traffic.

Conclusions.—In 1913, 85% of the operating expenses and 30% of the interest was due to traffic. Passenger service was more profitable than freight.

Excluding less than carload freight, 28% of the direct cost of handling freight was terminal, 33% was proportional to the resistance overcome in moving trains, and 39% was due to the distance the freight was moved. The cost of handling an empty car was about 75% of that of a load. Increasing the number of tons per car is more effective in reducing cost than increasing the number of cars per train.

With proper study of the problem and with intelligent selection and comparison of data, the cost of transporting commodities can eventually be computed as accurately as the cost of producing them.

Division of Operating Expenses into Direct and Indirect

The first step in the study of the cost of service is the segregation of those expenses which are not affected by the volume of traffic. They form a con-

stant charge against the existing business of the railroad, whether such business is large or small.

Some expenses are affected by variations in traffic expressed by one unit and not by others. For example, the cost of trainmen is affected by changes in train-miles, but not by car-miles. Traffic, as the word is used in this connection, is understood to be measured in the unit applicable to the account under consideration, and that the variations considered are in quantity only, without change in car or train loading, seasonal distribution, balance between opposite directions, or any other characteristic which would affect cost. This is necessary in order that the effect of variations in traffic may not be confused with the effect of changes in efficiency.

Only those variations in traffic which are of considerable extent, are taken into account. There are many minor variations in the volume of traffic, which have no perceptible effect on expenses. A few more passengers or tons of freight may be carried by the same equipment at an additional cost which is negligible, but with the growth of traffic, there comes a time when additional equipment and facilities must be provided, and more men and trains placed in service. There is then a sudden increase in cost of operation, which cannot properly be attributed solely to the last additional traffic that made it necessary, but which must be spread over all the business handled. Before such a change takes place, the operating cost per unit of traffic is less than the normal cost, and afterward, for a time, it is more than the normal cost. Both extremes are disregarded in this paper. Operating expenses respond much less readily to a decrease in traffic than to an increase, and the relations determined herein are not intended to apply to such conditions.

Certain operating expenses are partly indirect, because they are due, in part, to natural causes, and, in part, to the effects of traffic. This is true of nearly all the maintenance-of-way expenses. The roadbed and all the structures are affected in varying degrees by weather, rust, and decay, as well as by wear due to the wheels passing over them. The effect of natural causes remains the same under all circumstances, but its ratio to the total expenses decreases with the growth of traffic.

Transportation expenses are partly indirect from another cause. At the beginning of operations, a certain initial equipment is required, whether or not any traffic exists. For a time, this equipment is only partly used, and a large part of the corresponding expenses is indirect. When the growth of traffic passes the point where the initial facilities are fully used, additional facilities must be provided, approximately in proportion to the increase of traffic, and, thereafter, the expenses of both the maintenance and operation of the equipment are entirely direct. Indirect transportation expenses, therefore, would ultimately disappear, if it was not for the fact that, with a growing system, the process is repeated with each addition of mileage so that although these expenses are entirely direct on one part of the system, they are largely indirect on other parts.

This consideration affects also the direct proportion of maintenance expenses, for it is obvious that if the existence of facilities has been made neces-

sary by the growth of traffic, their maintenance is also a function of traffic. It is for this reason that the maintenance of industrial tracks is considered to be a direct expense.

By ascribing the expenses of maintaining and operating partly used facilities and equipment to direct cost only in proportion to the extent to which they are used, there is charged to direct cost only the cost of maintaining and operating equipment and facilities used to approximately normal capacity. By this, it is not meant that each train is loaded with all that it can carry. In order to take care of fluctuations in the volume of business, there must be operated, at all times, equipment and facilities in excess of the average requirements. When volume of traffic reaches the point at which the excess of capacity over average requirements is only sufficient properly to take care of peak loads, the equipment is being used to its normal capacity, and further increase will demand added equipment.

In estimating the direct and indirect proportions of operating expenses, the writer has considered the various primary accounts separately or in groups of related accounts. Many of the accounts are obviously direct or indirect, either wholly or in large part. Others can be estimated only on the basis of judgment and experience. In dealing with some of these accounts, the writer has made use of a method recommended by a committee of the American Railway Engineering Association* for comparing maintenance-of-way expenses, but has extended it to include certain other accounts.

In general, the method consists in calculating the density of traffic and the expense per mile in the account under consideration, and observing the extent to which the expense on railroads of heavy traffic exceeds that on those of light traffic. For this purpose, thirty American railroads have been selected, the majority of which are in the northern part of the United States and operate under conditions fairly comparable to those of the Grand Trunk Railway. Roads, the business of which is confined principally to freight or to passenger service, have been excluded.

The volume of traffic and the expense for each railroad for a given year was found from the records of the Interstate Commerce Commission. In each case, the roads have been divided into two groups, those above the average density of traffic and those below it. Assuming that the difference in expense in the two groups is proportional to the difference in traffic density, the percentage of the given expense due to traffic at any given density, has been calculated. By this means, all comparisons between large and small volumes of traffic are confined to the same period of time, thus avoiding the complications due to rising and falling prices and wages.

I.—Maintenance of Way and Structures

In applying the method of analysis described, the chief difficulty is in choosing a unit of traffic that is related closely enough to the expense under consideration, and in equating the units in freight and passenger service. The Committee of the American Railway Engineering Association, previously mentioned, recommends that for consideration of track-maintenance expenses,

* *Proceedings*, Am. Ry. Eng. Assoc., 1913.

what it calls "Equivalent Ton-Miles", shall be used to measure the density of traffic. That Committee estimates the damage, per ton-mile, to track caused by passenger trains as being twice that of freight trains, and the damage from locomotives as twice that of cars. Therefore, the passenger-locomotive ton-miles is multiplied by 4, the passenger-car ton-miles and the freight-locomotive ton-miles, by 2, and the products are added to the gross ton-miles of freight cars. The total is the equivalent ton-mileage. From consideration of a number of railroads, that Committee has derived a formula for determining the normal percentage of any expense which is direct for a given density of traffic, as follows:

Let X = amount of indirect maintenance per mile of track.
C = total maintenance per mile of track.
M = millions of equivalent ton-miles per mile of track.
I = increase in maintenance per mile for an increase of 1 in the value of M.
R = ratio of I to X.

Then, $X + M\,R\,X = C$, and $\frac{100\,M\,R}{1 + M\,R}$ = the percentage of C, which is direct. The equivalent ton-mileage of the Grand Trunk Lines in Canada for the year was found to be 21 585 000 000. On the basis of 3 118 miles of railroad, the value of M was 7, and on the basis of 3 825 miles of main tracks, it was 5.7.

The various primary accounts have been divided as follows:

6.—Roadway and Track.—A study of the statistics of the Interstate Commerce Commission for recent years, in which this account has been divided into two parts, shows that about 30% of it consists of the care of the right of way, which is not affected by adding more tracks. In compiling Tables 1 to 11, inclusive, the writer, therefore, has measured the density of traffic by dividing the equivalent ton-miles by the miles of main track, but for measuring the expenditure, 70% of the other main tracks has been added to the first, calling the sum "Equivalent First Main Track", and dividing this into the amount spent for maintenance. Side-tracks have not been included for the reason that investments in yard and industrial tracks are due to the increase in the traffic over that which existed when the road was built and, consequently, all maintenance expenses on those tracks must be considered as direct. The same thing is true, in part, of passing tracks. Table 1 shows the method of deducing a ratio from the records of the Interstate Commerce Commission. The equivalent ton-miles are calculated from the net ton-miles and the locomotive and car-miles.

The average density of traffic was 5.71 million equivalent ton-miles. The roads with a greater density had an average of 9.30 million, with an expense per mile of $778. Those with less density averaged 4.34 million and an expense per mile of $504. If a difference in traffic density of 4.96 million meant a difference in cost of $274, a difference of $M = 1$ would mean $55.

The part of the expense per mile of the second group, which was due to traffic, was $4.34 \times \$55 = \$239$. The remainder, \$265, was indirect, and is represented by X in the formula, whereas,

$$R = \frac{I}{X} = \frac{55}{265} = 0.21.$$

As $M = 5.7$, the direct percentage is,

$$\frac{100\,(5.7 \times 0.21)}{1 + (5.7 \times 0.21)} = 55\%.$$

The average for the three years, 1911, 1912, and 1913, was 60%, which was accepted as the true ratio. Fig. 1 illustrates Table 1.

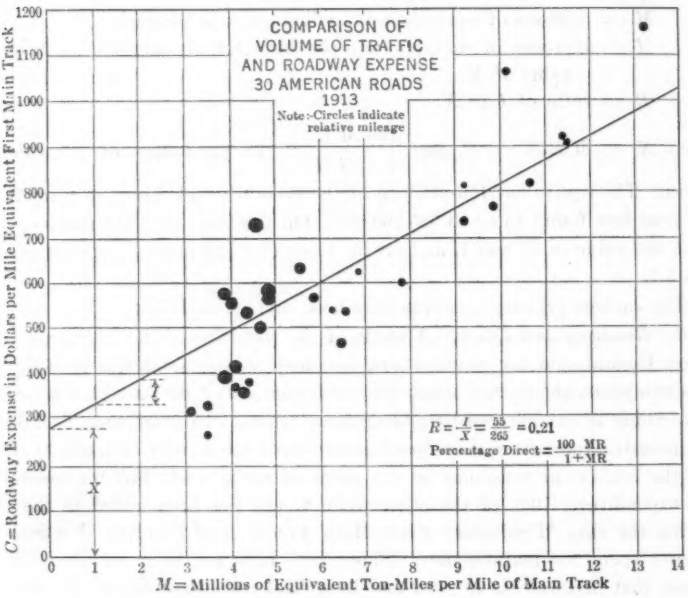


FIG. 1.

3.—Ties.—The cost and durability of ties varies so widely, and there are such differences in the practice of the various railroads in the preservation and protection of ties, that the writer has not relied on comparisons with other roads, but with the assistance of engineers familiar with the Grand Trunk Railway and the character of its maintenance, he has made estimates of the relative effects of traffic and natural causes on each section of the line. In this way, the tie maintenance of the System was classified as 34% direct.

4.—Rails and 5.—Other Track Materials.—In the case of these accounts, deterioration from any other cause than wear is so small that it may be ignored entirely, and the whole account treated as direct.

TABLE 1.—EFFECT OF VARIATIONS IN TRAFFIC ON ROADWAY AND TRACK MAINTENANCE, 1912.

Railroad.	Miles of railroad.	Miles of other main tracks.	Miles of equivalent first main track.	MILLIONS OF EQUIVALENT TON-MILES.		ROADWAY AND TRACK.	
				Total.	Per mile of main track.	Total.	Per mile of equivalent main track.
Pennsylvania	4 021	2 369	5 680	84 200	13.2	$6 608 000	$1 160
Philadelphia & Reading	1 015	620	1 450	18 800	11.5	1 322 000	910
Delaware, Lackawanna & Western	959	540	1 337	17 100	11.4	1 228 000	920
New York Central & Hudson River	3 595	2 338	5 232	62 200	10.7	4 303 000	820
Pittsburgh, Cincinnati Chicago & St. Louis	1 467	783	2 015	22 700	10.1	2 164 000	1 070
Lake Shore & Michigan Central	1 775	1 196	2 612	29 300	9.9	2 010 000	770
Philadelphia, Baltimore & Washington	713	356	962	9 800	9.2	779 000	810
Erie	1 988	977	2 672	27 300	9.2	1 972 000	740
Cleveland, Cincinnati, Chicago & St. Louis	2 011	426	2 310	19 200	7.9	1 375 000	600
Chicago & Alton	1 025	193	1 160	8 400	6.9	726 000	630
Michigan Central	1 817	610	2 244	17 700	6.6	1 201 000	540
Wabash	2 515	100	2 585	17 100	6.5	1 209 000	470
Vandalia	827	71	877	5 700	6.3	471 000	540
Illinois Central	4 763	926	5 410	33 300	5.9	3 076 000	570
Sub-totals	28 491	11 505	36 546	373 800	9.30	$28 439 000	$778
Southern Pacific	6 310	333	6 543	37 300	5.6	$4 194 000	$640
Atchison, Topeka & Santa Fe	8 201	827	8 780	44 000	4.9	5 179 000	590
Louisville & Nashville	4 740	172	4 860	24 200	4.9	2 744 000	560
Union Pacific System	7 257	884	7 876	38 100	4.7	3 930 000	500
Central Pacific	10 983	754	11 510	54 400	4.6	8 392 000	730
Chicago, Burlington & Quincy	9 074	785	9 624	43 100	4.4	5 086 000	530
Grand Trunk Western	1 496	105	1 570	7 100	4.4	584 000	370
Chicago, Rock Island & Pacific	7 566	290	7 770	33 900	4.3	2 700 000	350
Chicago & North Western	7 966	1 068	8 708	37 300	4.1	3 529 000	410
Chicago, St. Paul, Minneapolis & Omaha	1 744	129	1 834	7 600	4.1	659 000	360
Northern Pacific	6 420	624	6 857	28 500	4.0	3 831 000	560
Chicago, Milwaukee & St. Paul	7 511	680	7 987	32 200	3.9	3 111 000	390
Great Northern	7 482	201	7 623	29 700	3.9	4 401 000	580
Pere Marquette	2 330	276	2 523	9 100	3.5	682 000	270
Minneapolis, St. Paul & Sault Ste. Marie	3 773	59	3 814	13 000	3.4	1 226 000	320
Atlantic Coast Line	4 568	138	4 665	14 800	3.1	1 450 000	310
Sub-totals	97 415	7 325	102 544	454 800	4.34	$51 698 000	$504
Totals	125 906	18 830	139 090	828 100	5.71	.....	.....

9.—Bridges, Trestles, and Culverts.—The maintenance of culverts is not affected by traffic, and that of trestles very little. Steel bridges are very little affected by the elements, the principal indirect expense being for painting. The cost of retirements, which are due almost entirely to wear under traffic or to increase in weight of rolling stock, is direct. Records of the Grand Trunk Railway Company show retirements of steel bridges charged to operating expenses amounting to nearly one-half the total maintenance in 1913. The account, therefore, is estimated as 50% direct.

16.—Buildings, Fixtures, and Grounds.—Owing to the lack of any other basis for estimating the percentage of direct maintenance, recourse was had again to the records of other railroads. After much study, it was decided that equivalent ton-mileage although not directly affecting the maintenance of certain kinds of buildings, was, nevertheless, a sufficiently reliable standard by which to measure traffic density, and, accordingly, it was used in the same manner as it was for "Roadway and Track", except that miles of road and not of track were used for measuring both density and maintenance. For the three years, 1911-13, the records show an average value for R of 0.86. The value of M was 7.0, which makes the account 86% direct.

Other Expenses.—Of the remaining accounts under "Maintenance of Way", some are entirely indirect; some are small in amount, and have been assigned on somewhat arbitrary bases; and others are of a general nature and are apportioned according to the average of all accounts which can be directly assigned. They are combined in Table 2 as a single item.

II.—Maintenance of Equipment

The accounts under this group are nearly 100% direct. There is a slight reduction on steam locomotives, repairs, because of indirect train mileage (see Account 62, "Dispatching") and, in freight cars, repairs, a small allowance is made for decay.

III.—Traffic

This group of accounts was estimated to be 52% direct.

IV.—Transportation Expenses

The expenses in this group are most readily apportioned by segregating them first, as between freight and passenger and then segregating each, as between direct and indirect. Practically the entire group would be direct, if it was not for branch lines on which the traffic is below the capacity of minimum service permissible. At the beginning of operation, each district must be given an amount of service adequate for the needs of the public, which will require only a small number of short trains. For a long time thereafter, the growth of traffic is taken care of by adding cars, with no change in the number of trains, and, consequently, no effect on those expenses dependent on train mileage.

Certain accounts are variable, chiefly according to car mileage. As soon, therefore, as it becomes necessary to add more cars, those expenses become wholly direct. Fuel, water, lubricants, and other supplies for road locomo-

tives, are partly, but not proportionately, increased by that growth of traffic which can be cared for by additional cars alone, and do not become wholly direct until increased train mileage becomes necessary.

62.—Dispatching.—This expense is apportioned between freight and passenger service on the basis of train mileage; in accordance with the rules of the Interstate Commerce Commission, 56.3% is apportioned to freight.

It may be fairly assumed that as there was a large volume of travel on the main line, the passenger service was only that which was reasonably required to meet the demands of traffic, and that any substantial increase in traffic of the same character and having the same daily and seasonal distribution would have required additional train service. All main-line passenger-train expenses were, therefore, direct. The maximum number that could be averaged on branch-line passenger trains was taken as 4 cars, besides the baggage car; the actual average was 3.5 cars. These trains, therefore, were operated at 87.5% capacity, as it is fair to conclude that no more cars were hauled than the traffic required, although there is no doubt that the cars were only partly filled over much of the mileage. The two mileages were nearly equal so that of the two combined, 93.5% was direct, and this percentage was applied to the dispatching of passenger trains.

Freight trains are also of two classes:

1.—Through freight trains on the main line and important branches. It is assumed that through freight trains are not sent out until sufficient tonnage is available to make up full trains. All these expenses are, therefore, direct.

2.—Way freights and mixed trains, constituting the whole service on many branches and a part of it on the main line. The latter is estimated to be 100% direct, as it is all greater than the minimum amount required to give sufficiently frequent service, and the trains no doubt are as heavy as practicable, considering the equipment used, the grades encountered, and the nature of the service. Branch-line way-freight trains were found to haul about three-fourths as many cars per train as main-line way freights. The latter trains both discharge and pick up freight while moving in either direction, but a branch-line train does nearly all its unloading while moving from the main line, and its picking up, in the opposite direction. This accounts for a part of the difference in train loading; therefore, the branch-line way freights have been estimated to be 80% direct. Mixed trains were estimated to be 38% direct.

The total freight-train mileage was 11 026 000, classified as follows:

Through freight.....	8 762 000	miles,	100%	direct	=	8 762 000
Main line way.....	1 506 000	"	100%	"	=	1 506 000
Branch line way.....	446 000	"	80%	"	=	356 800
Mixed trains.....	312 000	"	38%	"	=	118 560
Total	11 026 000	miles,	97.4%	direct	=	10 743 360

This makes the total mileage 97.4% direct, which percentage is to be applied to all freight-train expenses. Together with the percentage of 93.5 for passenger trains, it makes the account of "Dispatching," 95.5% direct.

63.—Station Employees.—At small stations, a minimum force must be employed, but the traffic is often insufficient to keep such a force at work to full capacity. Usually, some additional traffic could be cared for at any station, without increasing the number of employees proportionately. Only an intimate acquaintance with the station work on every part of the System would enable one to make a reliable estimate of the extent to which this is true. The writer, therefore, has had recourse again to what for convenience may be called the "A. R. E. A. formula". The proper units for measuring growth of traffic for this purpose are the ton and the passenger, which has been equated on the assumption that 2 passengers equal 1 ton.

In 1912, thirteen of the roads given in Table 1, with an average traffic of 23.08 million "equated" tons per mile, spent $1 370 per mile for station employees. The corresponding figures for seventeen other roads are 4.38 million "equated" tons and $455 per mile for station employees. This gives a value for R of 0.20. Similar calculations give values for R equal to 0.21 and 0.19, respectively, for 1911 and 1913. On the same basis, the value of M is 8.5, and the percentage of direct, therefore, is 63. If 1 passenger is assumed as equivalent to 1 ton, the result will be changed only 1 per cent.

At some of the stations, the employees are directly assignable to freight or passenger service. These are estimated at 80% direct, and the remainder fixed at 53%, in order to make the total 63%, as previously given.

66.—Station Supplies and Expenses.—By the same method, this account is fixed at 73% direct.

67 to 76.—Yard Expenses.—These expenses are dependent on the number of cars handled, and the entire group of accounts, therefore, is considered to be 100% direct.

80.—Road Enginemen.—Direct assignment from the payrolls shows this account to be 70.6% freight and 29.4% passenger. Each account is direct in the ratio determined for train mileage under "Dispatching". This makes the entire account 96.3% direct.

82.—Fuel for Road Locomotives.—Fuel is assigned directly to freight and passenger service in the accounts of the Company, in the ratio of 70% freight to 30% passenger. The consumption of fuel while the train is in motion is directly proportional to the tonnage moved. A considerable quantity of the fuel is consumed by stops, road-switching, and waiting at stations. Such losses, however, will increase even faster than the increase in traffic, as the number of stations will be increased and more delays will be caused by trains meeting. The only coal consumption which is not direct, is a part of that accompanying train service not used to capacity. The coal consumption of way freights per train-mile was about the same as that of through freights, the fuel used in stops, waits, and road-switching making up for the lighter tonnage. Fuel consumption in freight service, therefore, was direct in the same ratio as train mileage, or 97.4 per cent.

Branch-line passenger trains were assumed to use three-fourths as much fuel per mile as main-line passenger-trains, so that the fuel consumption was

calculated to be 94.3% direct. Application of the ratios thus found, to freight and passenger service gives 96.5% of the entire account as direct.

Water, Lubricants, and Other Supplies.—These accounts are given the same classification as "Fuel".

88.—Road Trainmen.—Assignment direct from the payrolls shows the ratio in this case to be 72.3% freight and 27.7% passenger. Each service is direct in the ratio found under "Dispatching", namely, passenger, 93.5% direct, and freight, 97.4% direct. Applying these ratios, the entire cost is 96.3% direct.

89.—Train Supplies and Expenses.—The expenses in this group consist chiefly of cleaning, lighting, lubricating, and icing cars. Each expense is variable practically in accordance with the number of cars handled, therefore, the whole account is direct.

99.—Loss and Damage, Freight.—This account is entirely direct.

V.—General Expenses

These expenses are usually thought of as practically a fixed charge, but this is far from true. As traffic increases, employees are added in all general offices, new departments are instituted, and larger salaries are paid. These expenses have been apportioned by the writer on the average of all other Accounts 1 to 105, inclusive, which gives 85% direct.

Table 2 summarizes the analysis thus far, showing that, in 1913, 85% of the expenses of the Grand Trunk Railway Company was direct; that is, if the traffic had been 10% more or less, with the same character and distribution, the operating expenses would have been similarly changed 8.5 per cent.

The percentage of direct operating expenses is usually estimated at a much lower figure, but a study of the statistics of the Interstate Commerce Commission bears out these results. Indeed, if revenue is taken as the measure of tonnage, it will be found that the expenses of the larger carriers exceed those of the smaller ones in practically direct proportion to their traffic. The same conclusion must be drawn from the fact that operating ratios average approximately the same among roads of light and of heavy traffic. Revenue, however, does not accurately measure traffic, otherwise, one would be obliged to conclude that operating expenses are 100% direct.

Division of Operating Expenses Between Freight and Passenger Service

The rules of the Interstate Commerce Commission for the separation of operating expenses cover this subject so completely that the writer will not discuss it, except where he has departed from those rules.

I.—Maintenance of Way and Structures

Direct.—The rules of the Interstate Commerce Commission divide track-maintenance accounts on the basis of fuel used in the two services. In the present case, this rule would give 70% for freight, but the writer thinks that equivalent ton-mileage, as used by the American Railway Engineering Association, furnishes a better basis, and it has been used for all railroad tracks,

thereby assigning only 60% to freight. Maintenance of yard tracks and sidings is divided according to locomotive switching mileage, 90% freight. The different classes of tracks are equated, in maintenance expenses, for each account, as follows:

Ties:

1 mile of main track = 2 miles of yard tracks = 1⅓ miles of passenger tracks.

Rails:

1 mile of main track = 4 miles of yard tracks = 2 miles of passenger tracks.

Other Track Material:

1 mile of main track = 1 mile of yard track = 1 mile of passenger track.

Roadway and Track:

1 mile of main track = 3 miles of yard tracks = 2 miles of passenger tracks = 1.4 miles of second tracks.

The calculations are omitted for the sake of brevity.

Indirect.—The indirect maintenance-of-way expenses are not only independent of the volume of traffic, but also of its nature. There is nothing about the decay of a tie or the removal of snow from the track which attaches the expenses to freight as against passenger traffic, or the reverse. These expenses, therefore, cannot be apportioned on any special basis. They must be assigned arbitrarily, and the best basis for such assignment would seem to be that of the results obtained in each service. These results are indicated by the revenues, provided rates are properly adjusted as between the two services. This adjustment is most nearly correct when the earnings in each branch of the service bear the same relation to the direct cost in that branch. The indirect maintenance-of-way expenses are apportioned, therefore, in the same ratios as the total direct operating expenses, namely, 72.5% to freight, and 27.5% to passenger.

Table 3 shows the separation between freight and passenger service for both direct and indirect. The same ratios have been used for both, except as noted, or in certain transportation accounts, where the assignment is affected by the fact that one service is more completely direct than the other.

Direct Freight Operating Expenses, Division Between Terminal and Road

Definitions.—Terminal expenses are those incurred in the origination and delivery of traffic, without regard to the distance it is moved. They consist of loading, unloading and accounting for freight shipments, switching cars to and from industrial sidings, making up trains, in short, of all expenses by which freight is placed on the main-line track, ready to move, or taken from its position in the train to the consignee. They are divided into two classes, Car Load and Less than Car Load. All expenses incident to the movement of car loads of less than carload freight are included in the first class. The

second class contains only those expenses peculiar to less than carload. ("Terminal", as the word is used in this paper, does not refer to the ends of the road.)

TABLE 2.—ANALYSIS OF OPERATING EXPENSES, DIRECT AND INDIRECT, GRAND TRUNK RAILWAY OF CANADA, 1913.

Account.	Total.	Total.		
		Direct.		Indirect.
		Percentage.	Amount.	Amount.
I.—MAINTENANCE OF WAY AND STRUCTURES:				
3.—Ties	$1 014 237	34.0	$ 344 800	$ 669 437
4.—Rails	96 632	100.0	96 632	.....
5.—Other track material	203 855	100.0	203 855	.....
6.—Roadway and track	1 558 180	60.0	935 000	623 180
9.—Bridges, trestles and culverts	467 530	50.0	233 765	233 765
16.—Buildings	761 008	86.0	654 500	106 508
Other accounts	254 013	.....	36 307	217 706
Totals	$4 355 455	57.5	$ 2 504 859	$1 850 596
II.—MAINTENANCE OF EQUIPMENT:				
25.—Steam locomotives, repairs	$2 981 492	96.3	$ 2 871 000	$ 110 492
31.—Passenger cars, repairs	544 661	100.0	544 661	.....
34.—Freight cars, repairs	2 191 286	98.0	2 147 400	43 886
Other accounts	542 499	.....	468 800	73 699
Totals	$6 259 938	96.4	$ 6 031 861	$ 228 077
III.—TRAFFIC:				
All accounts	$1 217 251	52.0	$ 633 251	$ 584 000
IV.—TRANSPORTATION:				
62.—Dispatching	$134 257	95.5	$ 128 200	$ 6 057
63.—Station employees directly assigned	979 077	80.0	783 000	196 077
65.— " " apportioned	1 673 291	53.0	887 000	786 291
66.— " " supplies and expenses	263 106	73.0	192 000	71 106
67-76.—Yard expenses	2 599 712	100.0	2 599 712	.....
80.—Road enginemen	1 718 703	96.3	1 654 300	64 403
82-85.—Fuel, etc., for railroad locomotives	4 651 143	96.5	4 488 400	162 743
88.—Road trainmen	1 991 130	96.3	1 917 000	74 130
89.—Train supplies and expenses	551 853	100.0	551 853	.....
99.—Loss and damage, freight	401 432	100.0	401 432	.....
Other expenses	1 069 434	.....	950 272	119 162
Totals	$16 033 138	91.0	$14 553 169	$1 479 969
Outside operations	$261 364	75.0	$ 196 023	$ 65 341
V.—GENERAL EXPENSES	1 062 028	85.0	903 000	159 028
Grand totals	$29 189 174	85.0	$24 822 163	$4 367 011

Road expenses are those incurred in the moving of freight between the stations of origin and destination. They are of two kinds, namely, those proportional to the distance the freight is moved, called Distance Expenses, and those dependent on the physical resistance to movement, or Resistance Expenses. Distance is subdivided, according to the unit directly affected, into train-mile, car-mile, and gross ton-mile.

TABLE 3.—Analysis of Operating Expenses, Freight and Passenger.

Account.	Direct.				Indirect.			
	Total.	Freight.		Passenger.	Total.	Freight.		Passenger.
		Percentage.	Amount.	Amount.		Percentage.	Amount.	Amount.
I.—Maintenance of Way and Structures:								
3.—Ties	$ 344 800	70	$ 241 000	$ 103 800	$ 669 437	72.5	$ 485 000	$ 184 437
4.—Rails	96 632	62	60 000	36 632				
5.—Other track material	203 855	66	134 000	69 855				
6.—Roadway and track	935 000	64	600 000	335 000	623 180	72.5	452 000	171 180
9.—Bridges	233 765	60	140 000	93 765	233 765	72.5	169 500	64 265
16.—Buildings	654 500	70	458 000	196 500	106 508	72.5	77 208	29 300
Other accounts	36 307		18 700	17 607	217 706		157 980	59 726
Totals	$ 2 504 859	66	$1 651 700	$ 853 159	$1 850 596	72.5	$1 341 688	$ 508 908
II.—Maintenance of Equipment:								
25.—Steam locomotives, repairs	$ 2 871 000	71	$ 2 038 000	$ 833 000	$ 110 492	48	$ 53 000	$ 57 492
31.—Passenger cars, repairs	544 661	0		544 661				
34.—Freight cars, repairs	2 147 400	100	2 147 400		43 886	100	43 886	
Other accounts	468 800		329 450	139 350	73 699		44 295	29 404
Totals	$ 6 031 861	75	$ 4 514 850	$1 517 011	$ 228 077	62	$ 141 181	$ 86 866
III.—Traffic:								
All Accounts	$ 633 251	40	$ 250 018	$ 383 233	$ 584 000	48	$ 282 439	$ 301 561

TABLE 3.—(*Continued*)

Account.	Direct.				Indirect.			
	Total.	Freight.		Passenger.	Total.	Freight.		Passenger.
		Percentage.	Amount.	Amount.		Percentage.	Amount.	Amount.
IV.—Transportation:								
62.—Dispatching	$ 128 200	57.5	$ 73 700	$ 54 500	$ 6 057	34.0	$ 2 057	$ 4 000
63.—Station employees, assigned	783 000	89.5	701 000	82 000	196 077	89.5	175 154	20 923
63.—Station employees, apportioned	887 000	70.0	621 000	266 000	786 291	70.0	550 291	236 000
66.—Station supplies and expenses	192 000	53.0	102 000	90 000	71 106	53.0	37 700	33 406
67-76.—Yard expenses	2 599 712	90.0	2 340 000	259 712				
80.—Road enginemen	1 654 300	71.4	1 181 520	472 780	64 403	49.0	31 540	32 863
82.-85.—Fuel, etc., for road locomotives	4 488 400	70.8	3 177 700	1 310 700	102 743	48.0	78 230	84 513
88.—Road trainmen	1 917 000	73.0	1 400 000	517 000	74 130	53.0	39 400	34 730
89.—Train supplies and expenses	551 853	57.0	314 500	237 353				
99.—Loss and damage, freight	401 432	100.0	401 432					
Other expenses	950 272		606 985	343 287	119 162		59 985	59 177
Totals	$14 553 169	75.0	$10 919 837	$3 633 332	$1 479 969	66.0	$ 974 357	$ 505 612
Outside operations	$ 196 023	0	.....	$ 196 023	$ 65 341	0	.....	$ 65 341
V.—General Expenses	$ 903 000	72.5	$ 655 000	$ 248 000	$ 159 028	65.0	$ 103 000	$ 56 028
Grand totals	$24 822 163	72.5	$17 991 495	$6 830 758	$4 367 011	65.0	$2 842 665	$1 524 346

Costs are not necessarily classified by their causes, but by the factors in accordance with which they are variable. For instance, the expense of enginemen is incurred solely on account of Resistance, but is assigned to Distance, as the men are paid according to the miles run.

I.—Maintenance of Way and Structures

2 to 6.—Track Accounts.—Yard tracks and sidings are used for both origination and delivery of freight, which is a terminal expense, and, at division points and junctions, for operations connected with through movements, which belong to distance expenses. No accurate apportionment can be made between the two classes, but it can be approximated closely enough for the purpose of this paper.

The amounts of freight originated and delivered at each division point were known and from calculations of tonnage moved east and west, the tons into and out of each point were ascertained. The respective percentages of local and through tonnage were then calculated for each division terminal. The trackage at each point was next measured. After deducting the local tracks, which include team tracks and those serving freight-houses, stock-yards, warehouses, and industries, the remainder was divided between through and local service according to the percentages stated previously. In this way, the tracks of Grand Trunk Railway were classified, as follows:

	Miles.
First main track	3 118
Second main track	707
Passing tracks	231
Yard tracks used in through service	217
Yard tracks used in local service	182
Local tracks and sidings	597
Total	5 052

These figures were used for subdividing track maintenance expenses.

3.—Ties.—Of the direct expense which was apportioned to freight, 42% was on account of yard tracks and sidings. The maintenance of that part of those tracks which is used in local service is terminal, whereas that of the remainder, being proportional to the weight of cars handled, is charged to gross ton-miles. On account of less traffic over them, the writer has estimated that local tracks and sidings require only two-thirds as much tie maintenance per mile as yard tracks. The proportions are, therefore, as follows:

	Terminal.	Gross ton-mile.
Yard tracks in through service	...	217
Yard tracks in local service	182	...
Local tracks and sidings ($597 \times \frac{2}{3}$)	399	...
Total	581	217

The 42%, therefore, is divided into terminal carload, 31%, and gross ton-mile, 11 per cent. The remaining 58% is apportioned according to the equivalent ton-mileage in freight service, which was:

Locomotive ton-mileage	3 747 000 000
Gross ton-mileage	9 268 000 000
Total	13 015 000 000

The former belongs to resistance, 17%, and the latter to gross ton-miles, 41 per cent. The entire account, therefore, is apportioned:

Terminal carload	31%
Gross ton-miles	52%
Resistance	17%

The other track accounts are treated in the same manner, but the results vary, because of different values assumed for local as compared to yard tracks, which are, as follows:

Rails	1	mile	of	local	tracks	=	½	mile	of	yard	tracks
Other track material	1	"	"	"	"	=	½	"	"	"	"
Roadway and track	1	"	"	"	"	=	⅓	"	"	"	"

Because of variations in the proportions of maintenance due to yard tracks and sidings under each account, there is further difference in the results. The percentages apportioned to each classification are shown in Table 5.

9.—Bridges, Trestles, and Culverts.—These accounts were apportioned on equivalent ton-mileage, as was done with road tracks: Gross ton-mile, 71%, and resistance, 29 per cent.

16.—Buildings, Fixtures, and Grounds.—Station and office buildings are apportioned in the same manner as "Station Employees".

Roadway buildings are apportioned in the same proportion as "Track Accounts" and "Bridges", combined.

In the case of fuel stations, the ratio of direct freight fuel used in road service to all fuel charged to direct freight is 83 per cent. The same percentage of the maintenance of fuel stations is charged to resistance, and the remaining 17% is divided in the same manner as "Yard Expenses".

Water stations are apportioned in the same manner as fuel stations.

Shops are apportioned in the same manner as maintenance of equipment.

Engine houses are apportioned to resistance according to the proportion of engine-house expenses-road to total direct freight engine-house expenses, 73%, the remaining 27% to be divided the same as "Yard Expenses".

The apportionment of the various parts of the primary account gives, for the total: Terminal, less than carload, 27.8%; terminal, carload, 17.2%; train-miles, 3.2%; car-miles, 14.0%; gross ton-miles, 1.5%; and resistance, 36.3 per cent.

II.—Maintenance of Equipment

25.—Steam Locomotives, Repairs.—This expense is assigned according to locomotive mileage, whether road or yard. The former is 71% of the total, and this percentage, therefore, is charged to resistance. The remainder is

divided the same as "Yard Expenses," according to the switching mileage in terminal and through service.

34.—Freight-Train Cars, Repairs.—Mr. B. F. Zercher, former Master Car Builder at the Angus Shops of the Canadian Pacific Railway Company, and recently holding a similar position on the Grand Trunk Western Railway, who has given special study to the causes of damage to freight cars, estimates that 65% of all damage occurs in yard handling, 3% is due to curvature and grades, and the remainder is about equally divided between the starting and stopping of trains and causes independent of track or handling. Accepting his estimate (except for a small increase in amount due to resistance of curvature and grades), the writer has divided the yard handlings between Terminal and Distance in proportion to switching-locomotive mileage (see "Yard Expenses"). The apportionment of the entire account is as follows:

Handling local cars in yards (Terminal)	38%
Handling through cars in yards (Distance)	27%
Starting and stopping trains (Distance)	15%
Miscellaneous causes (Distance)	15%
Effects of curvature and grades (Resistance)	5%

The distance expense belongs to car-miles.

III.—Traffic

All expenses under traffic (Accounts 53 to 60), are terminal, and one-fourth has been apportioned to less than carload, and three-fourths to carload.

IV.—Transportation

62.—Dispatching.—This is entirely a train-mile expense.

63.—Station Employees.—The greater part of these employees are engaged in terminal operations, the principal exceptions being telegraph operators and men engaged in feeding and watering stock, straightening and reloading shipments, and other work connected with through business and the accounting therefor. These expenses are estimated as 15% of the entire account, which is divided between train-miles and car-miles.

Of the 85% terminal, the greater part belongs to less than carload business. Carload business requires only accounting, billing, weighing, occasional loading and unloading, and a proportion of the general expenses of the station. The writer estimates this as about one-sixth of the terminal charge. The account is apportioned as follows: Terminal, less than carload, 70%; terminal, carload, 15%; train-miles, 8%; and car-miles, 7 per cent.

66.—Station Supplies and Expenses.—These expenses are apportioned in the same manner as "Station Employees".

67 to 76.—Yard Expenses.—On account of greater average distance to be moved, smaller number of cars to be handled at one time, and the necessary movements of the empty car, 1 ton delivered to consignee, to team, or transfer track, or to freight-house, has been estimated as requiring as much switching service as 2 tons passing from one division to another. Weighting the local tonnage at this ratio, the switching-locomotive mileage at each point has been

apportioned between local and through service. The former required 58.6% and this percentage of the yard expenses, therefore, is charged to terminal carload. The remainder is assigned to car-miles.

80.—Road Enginemen.—Although enginemen are employed because of the resistance to be overcome, the expense on this account is variable in accordance with variations in the resistance only as far as it causes helper or assistant engines to be used. Freight-locomotive mileage exceeded freight-train mileage by 20.3%, therefore, this proportion is charged to resistance and the remainder to train-miles and terminal. The time spent in switching cars or waiting for less than carload freight to be loaded or unloaded is terminal. The percentage of time chargeable to each of these causes is estimated under Account 82 ("Fuel for Road Locomotives"), from which the following apportionment is made:

Train-miles (running)	42.0%
Train-miles (delays)	25.3%
Terminal carload (switching)	6.2%
Terminal, less than carload (waiting for less than carload)	6.2%
Resistance (helper service)	20.3%

82.—Fuel for Road Locomotives.—Fuel is consumed by road locomotives when engaged as follows: (1) Firing up; (2) pulling trains; (3) running down hill by gravity; (4) waiting at meeting points and terminal delays; and (5) waiting while freight is being loaded or unloaded.

Item (1) should be divided in the same proportions as the remaining uses and, therefore, may be omitted from this calculation; it amounts to about 3% of the total. Item (2) is clearly chargeable to resistance. Item (4) is indirectly dependent on resistance, as the greater the energy required, the more engine service is needed and the larger the waste of fuel in delays. It also varies with distance, as the longer the run the more time wasted in delays. The quantity of fuel thus consumed, however, is very small, and, therefore, the factor of distance is ignored herein. The fuel consumed while running down hill may be considered as an addition to that consumed while climbing the same grades, and charged to resistance. Fuel burned while less than carload freight is being loaded or unloaded, or cars picked up or set out, is a terminal expense.

In the following estimate of time consumed in various ways by freight-train locomotives, the percentage of time actually running is taken as 100, and is assumed to be the same for all trains. The velocity profiles of the main line from Chicago, Ill., to Portland, Me., show that, in each direction, 78% of the time is occupied in steaming and 22% in drifting. In making these calculations, it is estimated that an engine steams always at full capacity, shutting off steam entirely when the speed limit is reached, and drifting without steam until the speed is sufficiently reduced. The estimate of time consumed in other ways is based on observation and experience, but any probable error in it will not be large enough seriously to affect any conclusions reached in this paper.

The percentage distribution of time of freight-train locomotives on the road is as follows:

	Through freight.	Way freight.
Working	78	78
Drifting	22	22
Delays at terminals and on account of other trains	65	40
Switching at stations	..	80
Waiting account handling less than carload	..	80
Total	165	300

The average running time between stations, as shown by velocity profiles, was 23.0 miles per hour. The direct freight-train mileage shown on page 359, therefore, required in actual running time:

Through freight	8 762 000 miles	381 000 hours
Way and mixed freight	1 981 360 "	86 000 "

The entire time of these trains, distributed according to the ratios given, was:

Steaming:	Through	381 000 × 0.78	297 200	
	Way	86 000 × 0.78	67 100	364 300
Drifting:	Through	381 000 × 0.22	83 800	
	Way	86 000 × 0.22	18 900	102 700
Delays:	Through	381 000 × 0.65	247 700	
	Way	86 000 × 0.40	34 400	282 100
Switching:	Way	86 000 × 0.80	.....	68 800
Handling less than carload:	Way	86 000 × 0.80	.....	68 800
	Total			886 700

In percentage, this amounts to:

Running:	Steaming	41%	
	Drifting	11.6%	52.6%
Standing:	Delays	31.8%	
	Switching	7.8%	
	Handling less than carload	7.8%	47.4%
	Total		100.0%

The approximate consumption of coal, of 12 000 B. T. U. per lb., by locomotives not working, has been given,* as follows, in pounds per 1 000 sq. ft. of heating surface:

Firing up, per trip	510 lb.
Standing, per hour	121 lb.
Drifting, per hour	219 lb.

* *Bulletin 148*, Am. Eng. Assoc.

On the basis of 2 lb. of coal per hour per sq. ft. of heating surface, when steaming, the consumption while standing was 6% as much, and while drifting 11% as much. The fuel used while switching at stations is estimated at one-half as much per hour as when standing.

The percentages of fuel used in various ways are as given in Table 4.

TABLE 4.

	Percentage Rate:		Percentage of fuel.
	Of time.	Of use of fuel.	
Steaming	41.0	100% = 41.000	84.5
Drifting	11.6	11% = 1.276	2.6
Delays at terminals and account of other trains	31.8	6% = 1.908	3.9
Switching at stations	7.8	50% = 3.900	8.0
Handling less than carload	7.8	6% = 0.468	1.0
Total		48.552	100.0

This account, therefore, is apportioned: Terminal less than carload, 1%; terminal carload, 8%; and resistance 91 per cent.

Water, Lubricants, and Other Supplies.—These expenses are apportioned the same as "Fuel".

88.—Road Trainmen.—These expenses should be apportioned according to the time spent by engines in various operations, except that allowance should be made for part of a train crew being engaged in handling less than carload freight, while the remainder are switching:

Train-miles (running time plus waiting for other trains)	84.4%
Terminal carload (switching)	6.0%
Terminal less than carload	9.6%
Total	100.0%

89.—Train Supplies and Expenses.—All or nearly all these items are car expenses and are variable with the distance that freight is transported. The charge, therefore, belongs to car-miles.

99.—Loss and Damage, Freight.—To the extent that damages occur before freight is actually in transit, or after it has reached the station to which it is destined, they are a terminal expense. The writer estimates this proportion as 60%, of which two-thirds is less than carload. Of the 40% occurring in transit, 60%, is estimated as proportional to train-mileage. The entire account is: Terminal less than carload, 40%; terminal carload, 20%; train-miles, 24%; and car-miles, 16 per cent.

V.—General Expenses

These expenses are apportioned according to the proportions of all other accounts. Table 5 shows the complete analysis of direct freight expenses.

TABLE 5.—ANALYSIS OF DIRECT FREIGHT EXPENSES.

Account.	Direct Freight.												
		Terminal.				Road.							
						Distance.							
	Total.	Percentage.	Less than carload.	Percentage.	Carload.	Percentage.	Train-mile.	Percentage.	Car-mile.	Percentage.	Gross ton-mile.	Percentage.	Resistance.
I.—MAINTENANCE OF WAY AND STRUCTURES:													
3.—Ties	$241 000	....	.....	31.0	$75 000	....	.....	....	.....	52.0	$125 000	17.0	$41 000
4.—Rails	60 500	....	.....	5.5	3 300	....	.....	....	.....	68.0	40 800	26.5	15 900
5.—Other track material	134 000	....	.....	19.0	25 500	....	.....	....	.....	60.0	80 400	21.0	28 100
6.—Roadway and track	600 000	....	.....	12.0	72 000	....	.....	....	.....	64.0	384 000	24.0	144 000
9.—Bridges	140 000	....	.....	....	.....	....	.....	....	.....	71.0	99 400	29.0	40 600
16.—Buildings	458 000	27.8	$127 300	17.2	78 700	3.2	$14 700	14.0	$64 200	1.5	6 900	36.3	166 200
Other accounts	18 700	....	*6 000	....	*4 000	....	1 900	....	4 900	....	20 600	....	1 300
Totals	$1 651 700	7.3	$121 300	15.2	$250 500	1.0	$16 600	4.0	$69 100	46.0	$757 100	26.5	$437 100
II.—MAINTENANCE OF EQUIPMENT:													
25.—Steam locomotive repairs	$2 038 000	....	.....	17.0	$346 000	....	.....	12.0	$245 000	....	.....	71.0	$1 447 000
34.—Freight cars, repairs	2 147 400	....	....	38.0	816 000	....	.....	57.0	1 224 000	....	.....	5.0	107 400
Other accounts	329 450	....	.....	....	76 200	....	.....	....	127 650	....	$22 100	....	103 500
Totals	$4 514 850	....	.....	27.4	$1 238 200	....	.....	35.4	$1 596 650	0.5	$22 100	36.7	$1 657 900
III.—TRAFFIC:													
All accounts	$250 618	25.0	$62 500	75.0	$187 518	....	.....	....	.....	....	.....	....	.....

*Credit.

TABLE 5.—(*Continued*).

Account.	Direct Freight.													
		Terminal.				Road.								
						Distance.								
	Total.	Percentage.	Less than carload.	Percentage.	Carload.	Percentage.	Train-mile.	Percentage.	Car-mile.	Percentage.	Gross ton-mile.	Percentage.	Resistance.
IV.—Transportation:														
62.—Dispatching	$73 700	....	.....	....	.....	....	$73 700	....	.....	....	.....	....	.....
63.—Station employees	1 322 000	70.0	$925 000	15.0	$198 000	8.0	106 600	7.0	$93 000	....	.....	....	.....
66.—Station supplies and expenses	102 000	70.0	71 400	15.0	15 300	8.0	8 160	7.0	7 140	....	.....	....	.....
67-76.—Yard expenses	2 340 000	....	.....	58.6	1 371 000	....	.....	41.4	969 000	....	.....	....	.....
80.—Road enginemen	1 181 520	6.2	73 260	6.2	73 260	67.3	795 000	....	.....	....	.....	20.3	$240 000
82-85.—Fuel, etc	3 177 700	1.0	32 130	8.0	254 010	....	.....	....	.....	....	.....	91.0	2 891 560
88.—Road trainmen	1 400 000	9.6	134 000	6.0	84 000	84.4	1 182 000	....	.....	....	.....	....	.....
89.—Train supplies and expenses	314 500	....	.....	....	.....	....	.....	100.0	314 500	....	.....	....	.....
99.—Loss and damage, freight	401 432	40.0	160 600	20.0	80 300	24.0	96 300	16.0	64 232	....	.....	....	.....
Other expenses	606 985	....	*8 100	....	7 164	....	134 060	....	91 361	....	.....	....	382 500
Totals	$10 919 837	12.6	$1 358 290	19.0	$2 083 034	22.0	$2 395 220	14.2	$1 589 233	....	.....	32.2	$3 514 060
V.—General Expenses	$655 000	9.0	$59 000	21.7	$142 000	14.0	$92 000	18.5	$121 000	4.5	$30 000	32.3	$211 000
Grand totals	$17 991 405	9.0	$1 631 090	21.7	$3 901 252	14.0	$2 503 820	18.5	$3 325 983	4.5	$809 200	32.2	$5 820 060

*Credit.

Interest on Investment

Certain parts of the investment in railroad properties increase about in proportion with the increase of traffic. The principal items thus affected are equipment, buildings, yard and individual tracks, and second tracks. Interest on these investments is, therefore, a part of the direct cost, and has been distributed in the same manner as the operating expenses. The average cost of money to the Grand Trunk Railway Company has been estimated at 5%, including discount and commissions.

Equipment.—The investment in equipment was estimated, as follows:

Passenger locomotives	$ 3 736 000
Freight locomotives	10 651 000
Passenger cars	7 357 000
Freight cars	22 575 000

Locomotives.—The interest on investment in passenger locomotives is $186 800, of which, on the basis of passenger train-mileage, 93.5%, or $174 700, was direct. On freight locomotives, the interest was $532 550, of which 97.4%, or $518 700, was direct. This is divided on the basis of road and switching mileage, the same as locomotive repairs.

Passenger Cars.—Interest on this investment is $367 850, all direct.

Freight Cars.—The interest on this investment is $1 128 750, all direct, and is divided on the basis of time consumed in loading and unloading as compared with that in transit, which has been estimated as 60% and 40%, respectively. The amount, therefore, is apportioned as follows, terminal carload 60%, and car-mile, 40 per cent.

Buildings.—It was estimated that the investment in buildings was $17 000 000, and that the buildings were used to 90% of their capacity. The interest, therefore, was $850 000, of which $765 000 was direct. This is apportioned the same as the maintenance of buildings, fixtures, and grounds.

Yard Tracks and Industrial Sidings.—The investment in these tracks is estimated as 996 miles at $8 000 per mile, or roughly as $8 000 000, the interest on which would be $400 000. On the basis of locomotive switching, 90% belongs to freight. It has been shown that 217 miles of 996, or 22%, were used in through service. This percentage of the interest, therefore, is charged to distance, as a car-mile expense, and the remainder to terminal carload.

Second Tracks.—The Grand Trunk Railway Company has 707 miles of second track, but it appears that the greater part of this investment was not made necessary by the growth of traffic, and the inclusion of interest thereon as a direct charge on operations, therefore, would cause the relation of cost of service to growth of traffic to be incorrectly represented.

Hire of Equipment.—This item may properly be included with interest, as a charge of the same nature. The amount, $554 000, in the absence of records by which it could be directly assigned, has been divided between freight and passenger in the same ratio as the investment in cars, namely, 75% freight, 25% passenger, and the former has been apportioned in the same manner as interest on freight cars.

Indirect Interest.—In this study, as presented to the Board of Arbitration, only interest actually paid was considered, no account being taken of investment represented by stock. In order that the true relation between direct and indirect costs may be shown, the entire investment must be considered, as interest, or return on it, is as much a part of the cost of service as the operating expenses.

The books of the Grand Trunk Railway Company show on December 31, 1913, an investment in road and equipment of $380 000 000, or $122 000 per mile. Much of the property of the Grand Trunk was acquired in exchange for stock, so that the figures given do not represent the true investment. An approximate estimate is sufficient for the purpose of this paper, and, therefore, it has been taken at $250 000 000, or $80 000 per mile. The investment already assigned amounts to $70 000 000. The interest on the remainder is $9 000 000, which is divided between freight and passenger service in accordance with the sum of all direct costs, namely, freight, 73%, and passenger, 27 per cent. The results of the analysis of interest appear in Tables 6 and 7.

Taxes.—Taxes, amounting to about $475 000 per year, are an indirect cost and are apportioned on the same basis as indirect interest, namely, $347 000 to freight and $128 000 to passenger service.

Table 8 summarizes the cost of service. A comparison of earnings with cost in each service shows the passenger service to have been the more profitable. In 1913, freight revenues were $27 930 000, and passenger revenues, $14 280 000. The direct cost of freight service was 75% of the revenue, and that of passenger service was 55 per cent. Compared with the entire cost, the percentages were 110% for freight and 83% for passenger service.

Train-mile costs are shown to have been only about 60% as much as car-mile costs. A reduction of car-miles by heavier loading, therefore, will do more to effect economy than hauling more cars per train. The largest field for saving is in terminal operations which cost more than train-miles and car-miles combined. The direct cost was twice as much as the indirect cost.

Unit Costs

The direct cost of each unit of operation is next to be determined.

Terminal Less Than Carload.—The records of the Grand Trunk Railway Company afford no basis for estimating less than carload tonnage, but a similar analysis of cost of service of the Grand Trunk Western Railway Company gave 85 cents per ton for less than carload and one of the Atlantic and St. Lawrence Railway Company gave 80 cents per ton.

Terminal Carload.—The series of movements by which a car of freight is handled terminally has been classified, as follows:

At Point of Origin:

1.—Empty car from yard to shipper.
2.—Loaded car from shipper to yard.
3.—Loaded car from yard to train.

At Destination:

4.—Loaded car, train to yard.
5.—Loaded car, yard to consignee.

TABLE 6.—INTEREST ON INVESTMENT.

	Total.	TOTAL:		DIRECT:			
		Direct.	Indirect.	Percentage.	Freight.	Percentage.	Passenger.
Passenger locomotives	$186 800	$174 700	$12 100	....	.....	....	$174 700
Freight locomotives	532 550	518 700	13 850	....	$518 700	....	.....
Passenger cars	367 850	367 850	.....	....	.....	....	367 850
Freight cars	1 128 750	1 128 750	.....	....	1 128 750	....	.....
Buildings	850 000	765 000	85 000	70	535 500	30	229 500
Yard tracks and industrial sidings	400 000	400 000	.....	90	360 000	10	40 000
Hire of equipment	554 000	554 000	.....	75	415 500	25	138 500
Other interest	9 000 000	.....	9 000 000	....	.....	....	.....
Totals	$13 019 950	$3 909 000	$9 110 950	....	$2 958 450	....	$950 550

TABLE 7.—INTEREST ON INVESTMENT, DIRECT FREIGHT.

	Total.	TERMINAL:				ROAD:							
		Less than carload.		Carload.		Train-mile.		Car-mile.		Gross ton-mile.		Resistance.	
		Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.
Freight locomotives	$518 700	....	.....	17.0	$88 300	....	.....	12.	$62 400	....	.....	71.0	$368 000
Freight cars	1 128 750	....	.....	60.0	677 250	....	.....	40.	451 500	....	.....	....	.....
Buildings	535 500	27.8	$149 000	17.2	92 000	3.2	$17 100	14.	75 000	1.5	$8 000	36.3	194 400
Yard tracks and industrial sidings	360 000	....	.....	78.0	281 000	....	.....	22.0	79 000	....	.....	....	.....
Hire of equipment	415 500	....	.....	60.0	249 300	....	.....	40.0	166 200	....	.....	....	.....
Totals	$2 958 450	5.0	$149 000	46.9	$1 387 850	0.6	$17 100	28.2	$834 100	0.3	$8 000	19.0	$562 400

TABLE 8.—Cost of Service

Direct	$28 731 163	67.3%	
Indirect	13 952 961	32.7%	
Total	$42 684 124		
Freight	$30 790 520	72.1%	
Passenger	11 893 600	27.9%	
	$42 684 124		

	Direct.				Indirect.			
	Freight:		Passenger:		Freight:		Passenger:	
	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.
Operating expenses	72.5	$17 991 405	27.5	$6 830 758	65.0	$2 842 665	35.0	$1 524 346
Interest	75.7	2 958 450	24.3	950 550	73.0	6 651 000	27.0	2 459 950
Taxes	.....	.....	.....	.....	73.0	347 000	27.0	128 000
Totals	73.0	$20 949 855	27.0	$7 781 308	70.5	$9 840 665	29.5	$4 112 296

Direct Freight

	Terminal.				Road.							
	Less than carload.		Carload.		Train-mile.		Car-mile.		Gross ton-mile.		Resistance.	
	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.	Percentage.	Amount.
Operating expenses	9.0	$1 631 090	21.7	$3 901 252	14.0	$2 503 820	18.5	$3 325 983	4.5	$809 200	32.3	$5 820 060
Interest	5.0	149 000	46.9	1 387 850	0.6	17 100	28.2	834 100	0.3	8 000	19.0	562 400
Totals	8.5	$1 780 090	25.3	$5 289 102	12.0	$2 520 920	19.8	$4 160 083	3.9	$817 200	30.5	$6 382 460

6.—Empty car, consignee to yard.

7.—Empty car, yard to train (for return).

At Point of Origin:

8.—Empty car, train to yard.

All these operations are considered as equal in cost, and the sum of them is taken as the number of units of terminal carload operation. This apportionment is based on handling by yard crews, but practically the same work is done at other points by road engines. The cost of maintenance of all tracks and equipment, all accounting work, and the various switching movements not directly assignable to any of the operations previously mentioned, are considered as spread uniformly over them.

In computing the number of these movements, the writer has considered interchange of traffic with the lines of the Grand Trunk Railway Company in the United States as not equivalent to terminal handling, but to movement through division points, which is a car-mile expense. The total tonnage handled is classified as follows, the number of cars being calculated at the ratio of 18.18 tons per car, the average loading of loaded cars at that time:

	Tons.	Cars.
Local Tonnage:		
Originated	6 732 000	370 400
Delivered to consignee	11 980 000	659 000
Interchange:		
Received from carriers	8 979 000	493 900
Delivered to carriers	4 886 000	268 700
Totals	32 577 000	1 792 000

Each car of local tonnage originated required the first three movements designated previously, or $3 \times 370\,400 = 1\,111\,200$ unit terminal movements. Each such car delivered required the fourth to sixth movements, or, $3 \times 659\,000 = 1\,977\,000$ units.

In interchange, the connecting road takes the place of the shipper or consignee, but there is no empty car to be placed or taken away. Each interchange, therefore, required two unit movements instead of three, making 1 525 200 units.

In 1913, empty car-miles amounted to 85 000 000. The average haul of loads was 178 miles, and this has been taken as the distance moved by each empty car. There were then 477 500 empty cars moved, each of which required the seventh and eighth movements, making 955 000 unit movements. The total was 5 568 400 unit movements.

The direct cost is shown by Table 8 to have been $3 901 252, or 70 cents per unit for operating expenses, and $1 387 850, or 25 cents per unit for interest, a total of 95 cents per unit of terminal handling.

Distance.—Train-Miles.—The operations of the Grand Trunk Railway Company for 1913 required 10 743 360 direct freight train-miles, at a cost for expenses peculiar to train movements of $2 520 920, or 23.5 cents per train-mile, practically all operating expenses.

Distance.—Car-Miles.—The freight car-miles amounted to 300 331 000, at a cost of $3 325 983, or 1.11 cents per car-mile, for operating expenses, and $834 100, or 0.28 cents for interest, a total of 1.39 cents per car-mile.

Distance.—Gross Ton-Miles.—There were 8 440 727 000 gross ton-miles produced at a cost of $817 200, or 9.7 cents per 1 000 ton-miles, practically all for operating expenses.

Resistance.—The unit of Resistance is that of lifting 1 ton 1 ft., or of 1 ft-ton. The results of the experiments at the University of Illinois have been used in making velocity profiles. Calculations based on an assumed average weight of 35 tons per car, have been made showing the resistance necessary to be overcome in moving a train over each district of the System, making due allowance for acceleration, speed, grades, curvature, and stops, and tables have been prepared giving the foot-tons of work actually done in moving freight on the railroad during the period under consideration. The total resistance overcome was 275 454 000 000 ft-tons. The cost was $5 820 060, or 2.11 cents per 1 000 ft-tons for operating expenses, and $562 400, or 0.2 cents per 1 000 ft-tons for interest, a total of 2.31 cents per 1 000 ft-tons.

Indirect Cost.—About 8.5% of the direct cost is apportioned to terminal handling of less than carload freight. The same proportion of the indirect (freight) cost, or $830 000, therefore, is assigned to less than carload freight. About 25% of the direct cost is assigned to terminal carloads, which is independent of the distance freight is moved. The writer, therefore, apportioned 25% of the indirect cost, or $2 460 000, on the basis of tons handled terminally. This equals 7.5 cents per ton of carload freight. The remainder, $6 350 665, is apportioned on the basis of net ton-miles, amounting to 0.17 cent per ton-mile.

Similar analyses were made of the operations of the Grand Trunk Western Railway and the Atlantic and St. Lawrence Railway. The unit costs of all three roads were found to be as given in Table 9.

TABLE 9.

Railroad.	Cost of less than carload per ton, in cents.	Cost per carload per unit of operation, in cents.	Cost per train-mile, in cents.	Cost per car-mile, in cents.	Cost per 1 000 gross ton-miles, in cents.	Cost of resistance per 1 000 ft-tons, in cents.
Grand Trunk.....	.....	0.95	23.5	1.39	9.7	2.31
Grand Trunk Western.	84.7	1.14	24.7	1.25	8.65	1.98
Atlantic and St. Louis.	80.0	0.72	28.1	1.24	12.00	2.40

The details making up each unit cost for the Grand Trunk Railway Company are given in Table 10.

Operating Conditions

The first step in the application of unit costs is the determination of the exact conditions of train and car loading under which traffic was moved. In applying the results of the calculations of resistance to cars of weights other

than the average of 35 tons, allowance must be made for the corresponding difference in frictional resistance. The correction does not apply on descending grades exceeding about 0.3%, as the resistance is then overcome by gravity.

TABLE 10.—DETAILS OF UNIT COSTS.

Account.	Terminal carload, in cents.	Train-mile, in cents.	Car-mile, in cents.	1 000 gross ton-miles in cents.	1 000 ft-tons, in cents.
MAINTENANCE OF WAY:					
Track expenses	3.2	.....	.....	9.00	0.10
Buildings	1.3	0.11	0.021	.....	0.06
Miscellaneous expenses	.....	0.01	0.002	.....	0.01
MAINTENANCE OF EQUIPMENT:					
Locomotives, repairs	6.2	.....	0.082	.....	0.53
Freight cars, "	14.6	.....	0.408	.....	0.04
Miscellaneous expenses	1.3	.....	0.043	0.26	0.04
TRAFFIC	3.3	.....	.....	.....	.....
TRANSPORTATION:					
Dispatching	.....	0.68	.....	.....	.....
Station employees	3.5	1.00	0.031	.....	.....
" supplies and expenses	0.3	0.08	0.002	.....	.....
Yard expenses	22.2	.....	0.292	.....	.....
Joint yards and terminals	.....	.....	.....	.....	.....
Road enginemen	1.3	7.32	.....	.....	0.09
Engine house expense, road	0.4	.....	.....	.....	0.08
Fuel, road	4.3	.....	.....	.....	0.98
Water, etc., road	0.3	.....	.....	.....	0.06
Road trainmen	1.5	11.10	.....	.....	.....
Train supplies and expenses	.....	.....	0.105	.....	.....
Clearing wrecks	.....	0.36	0.009	.....	.....
Loss and damage, freight	1.4	0.90	0.022	.....	.....
Damage to stock	.....	0.08	.....	.....	.....
Miscellaneous expenses	2.2	0.82	0.052	.....	0.05
GENERAL EXPENSES	2.6	0.86	0.040	0.36	0.08
INTEREST:					
Freight locomotives	1.6	.....	0.021	.....	0.13
" cars	12.1	.....	0.150	.....	.....
Buildings	1.7	0.16	0.025	0.10	0.06
Yard and industrial tracks	5.0	.....	0.026	.....	.....
Hire of equipment	4.5	.....	0.055	.....	.....
Totals	94.8	23.51	1.386	9.72	2.31

Empty-Car Movement.—On all districts of the Grand Trunk Railway, there is throughout the year a preponderance of traffic eastbound. There is a large empty-car movement westward and a small one eastward. The cost of each belongs to the freight moved in the opposite direction. Where the traffic is of special character, requiring the return west, empty, of each car sent east, the calculation of cost is made accordingly. Otherwise, each loaded car east is charged with a fraction of an empty car west according to the ratio of empties to loads. The train-mile and car-mile costs of empty-car movements are the same as for loads. The gross ton-mile cost varies with the weight. The resistance cost is the same per foot-ton, but the resistance varies with the weight of the car.

Train Loading.—It is assumed that the ratio of car-miles to train-miles eastbound on each division represented the normal train loading for that division. The loaded car-miles westbound were divided by this ratio, in order to

obtain the train-miles required to move west bound freight. The remaining actual train-miles west were considered as belonging to the empty movement, and the number of empties per train was determined accordingly. It is true that loaded and empties moving westward are not separated in the manner described, but it appears to be improper to charge to the cost of west bound freight any expense arising from its being mixed with empties hauled west in order to carry freight east.

Application of Unit Costs

The cost per car for a given freight movement is equal to the sum of the following:

The terminal carload cost per car, at each end of the movement.

The train-mile unit cost, multiplied by the distance moved, and divided by the number of cars per train.

The car-mile unit cost, multiplied by the distance moved.

The gross ton-mile unit cost, multiplied by the distance moved, and by the number of gross tons per car.

The resistance cost per foot-ton, multiplied by the number of foot-tons resistance per ton, and by the gross weight of the car, plus a proportionate part of the weight of the engine and the caboose.

Charge for empty-car movement in opposite direction, if any.

Table 11 shows the application of the unit costs to the movement of a car of dressed meats from Chicago to Portland, Me. The average load carried was 10 tons, with 3 tons of ice, making the gross weight 38 tons. The number of cars and of gross tons per train on each district were taken from tables compiled for the purpose from statistical records of the Grand Trunk Railway. The equivalent rise in feet, under resistance, was taken from velocity profiles, corrected for a 38-ton car.

The terminal cost at Elsdon, Ill., is the unit cost of the Grand Trunk Western Railway multiplied by three, and covers the movements required to place the car in a train ready to leave the Elsdon Yard. This cost is assumed to be the same at all stations on the railroad. Probably some modifications should be made for special conditions at Elsdon, but this was not done, for the effect would be relatively small on so long a haul, and the exact conditions were not known. At Deering, Me., the rates of the Atlantic and St. Lawrence Railway Company were used.

The length of each district, divided by the number of cars per train, gives the number of train-miles, the cost of which must be borne by each car. The total tons under resistance equal the weight of the car, 38 tons, plus a proportionate part of the weight of the engine and tender, taken as 145 tons. Cars are handled from the stockyards to Elsdon, from Mimico, Ont., through Toronto, to York, Ont., Canada, and from Deering to Portland, Me., by yard engines, and the cost is included in yard expenses.

Certain corrections must be made if cars are owned by the shipper. Charges for freight-car repairs, train supplies, interest on freight cars, and

TABLE 11.—DIRECT COST OF TRANSPORTATION, CHICAGO, ILL., TO PORTLAND, ME., OF ONE CAR OF DRESSED MEATS, WEIGHING 10 TONS NET, 38 TONS GROSS.

District.	Miles.	Train.		Terminal Carload.		Train-Miles.			Car-Miles.			Gross Ton-Miles.			Resistance.					Total.
		Cars.	Gross tons.	Rate per unit.	Amount.	Number.	Rate, in cents.	Amount.	Number.	Rate, in cents.	Amount.	Number.	Rate per mile, in cents.	Amount.	Total tons.	Equivalent rise, in feet.	Foot-tons.	Rate per mile, in cents.	Amount.	
Grand Trunk Western Railway:																				
Elsdon-Port Huron	324	34.5	1 044	$1.14	$3.42	9.4	24.7	$2.32	324	$1.25	$4.05	12 300	8.65	$1.07	43.3	6 250	271 000	1.98	$5.36	$16.22
Grand Trunk Railway:																				
Sarnia to Mimico	161	34.1	1 089	....	....	4.7	23.5	1.10	161	1.39	2.24	6 120	9.70	0.60	43.0	3 570	153 600	2.31	3.55	7.49
York to Turcot	325	39.7	1 228	....	....	8.2	23.5	1.93	325	1.39	4.52	12 350	9.70	1.20	42.5	7 500	318 500	2.31	7.35	15.00
Turcot to Norton Mills	132	31.5	1 060	....	....	4.2	23.5	.98	132	1.39	1.83	5 000	9.70	0.48	43.2	4 100	177 000	2.31	4.09	7.38
Atlantic and St. Lawrence Railway:																				
Norton Mills to Deering	165	32.4	1 133	0.72	2.16	5.1	28.1	1.43	165	1.24	2.05	6 270	12.0	0.75	42.9	3 220	138 000	2.40	3.31	9.70
Totals	1 107	35.3	....	....	$5.58	....	....	$7.76	....	....	$14.69	.....	.....	$4.10	.....	.....	.....	.....	$23.66	$55.79

Deductions on account of privately owned cars..... $1.95	$8.45	
Deductions on account of carriage as manifest freight	3.11	
Total deductions		$ 13.51
Additions: Rental for car, 1 121 miles at $0.01	$11.21	
Transfer through St. Clair Tunnel	1.00	
Total additions		12.21
Net total		$34.49

hire of equipment, as far as it has been applied to cars at an average rate, must be eliminated.

Another correction is to be made because dressed meats move in manifest, or fast freight, trains, which are made up so as to require less handling in yards than other trains; therefore, the charge for yard expenses, under car-mile costs, must be reduced for this class of business. The charge is estimated by the writer at one-fourth as much as for ordinary through freight, and the manifest is estimated as one-fourth the total through movement. Locomotive repairs would be similarly affected, as all this expense which is charged to car-mile costs is on account of through cars in yards. Of freight-car repairs, about one-half come from the same cause, and would be affected in the same ratio. The results of these corrections are shown in Table 11.

Additions to the calculations mentioned are made because of rentals paid for the use of privately owned cars, 1 cent per mile, whether loaded or empty, and for passage through the St. Clair Tunnel, $1 for the loaded car and 50 cents for the empty.

An empty car hauled in the same train, would have one-third as much for terminal cost, and the same cost for train-miles and car-miles. The gross ton-miles would be proportional to the weight of the car. The "total tons", under resistance, would also be proportionately reduced, but the resistance per ton would be about 50% more for the light car. The total cost would be about 80% as much as for the load. The cost of the return of the empty refrigerator car to Chicago was calculated at $50.40, the more than average weight of the car and the charge for rent bringing the cost, in this case, to 90% of that of the load.

The full cost of the return must be charged to the east bound freight because nothing was moving west for which this type of equipment was suitable. In a similar calculation for a commodity moving in general purpose cars, there should be included only that fraction of the cost of the empty car represented by the ratio of the empty cars west to the loads east. If, however, the question, instead of being one of distribution of costs among kinds of business already existing, was one of the cost of taking on additional traffic eastward, it would be proper to charge against it the entire cost of the empty-car movement west. On the other hand, additional traffic westward would not only have no empty movement against it, but the additional cost would be only the difference between the cost of moving a load and that of an empty, which would be small, as shown previously.

The indirect cost has not been computed for the Grand Trunk Western and the Atlantic and St. Lawrence Railways, owing to the difficulty of determining the investment in these subsidiary companies, but it may be taken as the same as that of the Grand Trunk Railway. The amount assigned to each ton of the freight under consideration would be:

1 ton at Elsdon, Ill.	$0.075
1 121 ton-miles @ $0 017	1.900
1 ton at Deering, Me.	0.075
Total	$2.050

The application of unit costs thus ascertained will always require care in allowing for the special conditions surrounding each case, but the writer believes that it will be found practicable to apportion costs of handling various commodities with an accuracy comparable to that of a manufacturer calculating the cost of the various products of his factory. Discussion is needed as to the principles set forth and the ratios used herein, in order that a wider range of experience and judgment may be brought to bear on them.

In the preparation of this analysis, other members of the Arbitration Staff of the Grand Trunk Railway Company were consulted repeatedly. Valuable assistance was rendered by Messrs. R. A. C. Henry, Engineer in Charge, and M. A. Fullington and F. W. Cooper, Division Engineers. Messrs. George O. Somers, Traffic Expert, and A. D. Porter, Mechanical Engineer, furnished much useful information. Mr. V. I. Smart and E. W. Moreland, Assoc. M. Am. Soc. C. E., who were assigned as Assistants on the work, were invaluable in assembling material and in the solution of the many problems which arose. The criticisms of J. G. Sullivan, M. Am. Soc. C. E., on the completed work caused much of it to be revised, and, it is believed, to be greatly improved.

The science of railroading covers too wide a field for any one man to be expert in every part of it. The writer was especially fortunate in having the advice and assistance of so many men of wide experience and ripe judgment. If the results prove useful in the solution of railroad problems, a large share of the credit will be due to these men.

DISCUSSION

J. G. SULLIVAN,* M. AM. SOC. C. E. (by letter).—The author is deserving of the gratitude of railway engineers for bringing this important subject before the public. There is every reason why railroad men should know what transportation is really costing, and, further, the students of the economics of railway location will be well advised in giving this paper very careful study, although they may not agree with all the methods and the indicated conclusions.

In general, the writer's criticisms of the paper resulted in decreasing the ratio of the direct charges to the total.

The reason for this further discussion is that the writer believes that in reading Mr. Newell's paper those who are not familiar with railway operation may arrive at a false conclusion, namely, that, in general, passenger business is more profitable than freight business.

It is not thought that Mr. Newell intended to give that impression, but the writer's experience has been that there are certain classes always ready to grasp at straws to further their arguments as to why passenger rates should be lowered.

In 1894, the writer listened to a member of the "Coxey Army" haranguing a crowd in St. Paul, Minn., whose grievance was high passenger rates. He quoted the cost per 100 lb. weight for carrying a hog from St. Paul to Chicago, and compared this rate with the fare he would have been charged by the railway. One can readily see now such a man would take undue advantage of any figures that might tend to prove his case, especially if he were a member of a legislative body.

The writer cannot agree with the author's conclusion that "passenger service was more profitable than freight", even in the particular case under consideration. The passenger business on the Grand Trunk Railway was the "star boarder", as it is in all railways. The track has to be kept in shape for that particular business. Extra side-tracks of extra lengths must be built so that the freight business can be kept entirely out of the way of its more favored rival. If these facts are not taken into consideration, then, in the writer's opinion, the conclusions are not reliable.

A few of these adverse factors that increase the cost of freight service may be mentioned in the following questions:

(1) If a road had no passenger service, how much could it reduce the first cost and interest of locomotives and cars on account of being able to make better time?

(2) How much could the number and length of passing sidings be reduced, if there was no passenger service? (A double-track railway would practically require none.)

(3) It is true that Mr. Newell loads the passenger traffic in computing the cost of maintenance-of-way charges, but how many railway engineers will admit that this loading more than compensates for the actual damage done

* Cons. Engr., Winnipeg, Man., Canada.

due to speed, leaving nothing to compensate for keeping the roadway in a condition far better than the economics of freight service would warrant.

(4) How much of the fuel used in stopping and starting freight trains and burned while standing in side-tracks waiting for passenger trains to meet or pass, should be charged to passenger service?

(5) How much of the extra time of train crews and engine crews in freight service, as mentioned in Factor (4), should be charged to passenger service?

(6) Even if it is taken for granted that Mr. Newell's figures for the ratio of direct charges are correct, is he justified in dividing taxes, interest, and general expenses in the same ratio?

Most business concerns will compare the direct charges with the revenue from a certain line and then decide whether or not it is profitable to continue that line. One might get the impression from the author's statement that, "compared with the entire cost, the percentages were 110% for freight and 83% for passenger", that it would be advisable for the Grand Trunk Railway Company to go out of the freight business; such a conclusion, of course, would be absurd. If Mr. Newell had divided what he calculated as indirect interest, taxes, and general expenses on the basis of revenue received, the result would have been 107% for freight service and 90% for passenger service. If these items had been divided "fifty-fifty", the result would have been. 101% for freight service and 100% for passenger service.

The writer does not suggest that these latter divisions are more logical than those used by Mr. Newell, but simply uses them to show what very different results one may obtain, depending on the point of view that suggests the assumptions.

Another feature that supports the writer's views is the fact that although it would have been to the interest of the Government to establish Mr. Newell's conclusion, the solicitors did not feature this conclusion.

The writer wishes in no way to detract from Mr. Newell's excellent paper other than to prevent those who may not be familiar with railway operation making a mistake by drawing false conclusions.

J. P. Newell,* M. Am. Soc. C. E. (by letter).—The question raised by Mr. Sullivan in regard to the apportionment of the indirect expenses between freight and passenger service is one which confronts the student in every allocation of costs of production. These expenses would exist to the same extent if the operations of the road were confined entirely to either service. If the problem was the advisability of abandoning one branch of service, the passenger service, for example, it would then be proper to charge the entire amount of indirect cost to freight service, with the result that passenger business would appear to be tremendously profitable. In like manner, a large profit could be shown for freight service by assuming that the primary function of a railroad is to carry passengers, and charging all indirect costs to that service.

The writer has assumed that it is the business of a railroad to carry both freight and passengers and that neither should have any preference in alloca-

* Cons. Engr., Portland, Ore.

tion of charges. It is necessary, therefore, to find some equitable basis for apportioning those costs which are not dependent on either the volume or the character of the traffic carried. Just what that basis should be, it is not easy to determine. Apportionment according to the respective revenues at once suggests itself. This method, however, is correct only if freight and passenger charges are adjusted properly with reference to each other. The writer has not been able to find any more equitable basis than that which he has used.

In the matter of direct costs, only the maintenance-of-way charges offer any serious difficulty in allocation. Even if the clearly unreasonable proportion of 75% of these expenses on the Grand Trunk Railway in 1913 was charged to passenger service, such service would still show a slight gain, and freight, a loss. It appears, therefore, to the writer, that although there is much room for difference of opinion as to the exact method of distributing costs between freight and passenger service, the latter was the more profitable in that year. Since then, however, the comparative falling off in Grand Trunk passenger traffic has undoubtedly changed this relation.

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PROGRESS IN HIGHWAY RESEARCH

By W. K. Hatt,* M. Am. Soc. C. E.

It appears that an early solution of the more complicated problems, and, from the standpoint of National well-being, the graver problems of highway transport, will be reached only in so far as the highway engineer, the highway official, the vehicle designer and manufacturer, and the organizer and operator of highway transport systems, in conference, can come to an understanding of the aims of each, and to a recognition of the public service to which they are called. To this group should also be added, steam and electric railway operators, so that highway transport may find its appropriate function in a co-ordinated structure of National transportation.

Highway engineers and motor vehicle manufacturers are now seeking a common viewpoint, and are in the way of finding a common purpose from which to reach sound policies in the financing, building, and the use of highways. It is hoped that the contribution of those representing other systems of transportation to a scientific study of these questions may be had.

Co-Ordination of Research.—The Advisory Board on Highway Research, Division of Engineering, National Research Council, is organized to prepare a comprehensive National program for highway research; to assist existing organizations to co-ordinate their activities therein; and to collect and distribute information of completed and current research. This is not a directing, but a service, organization, devoted to the interest of the individual researcher. Services that can be rendered the individual researcher are in soliciting support for his projects, in putting him in touch with his fellow workers through a Census of Research, and in illuminating a particular field with sidelights from allied fields in which a project has its setting. The taxpayer will also benefit through a co-ordinated research activity on problems selected from a comprehensive program with reference to the applicability to present-day problems.

For example, highway researches in the field of economics of operation are connected under Committee No. 1 on the Economic Theory of Highway Improvement, T. R. Agg, M. Am. Soc. C. E., Chairman. In New England, the tractive resistance of automobiles and trucks is being measured on several types of roads at speeds up to 30 miles per hour. The vehicles are tested for internal absorption of energy at the Yale Laboratory by Professor E. H. Lockwood, and the gross tractive resistance determined on the roads by Maj. Mark L. Ireland, Q. M. C., U. S. A., who is stationed at the Massachusetts Institute of Technology. Co-operating in this work are: The Bureau of Public Roads, U. S. Department of Agriculture, the Committee on Economic Theory of Highway Improvement, National Research Council, the Connecticut Highway Commission, the Department of Public Works, Commonwealth of Massachusetts, Harvard University, the Massachusetts Institute of Technology, the Society of Automotive Engineers, the United States Army, Quartermaster Corps, and Yale University.

* Director, Highway Research Committee, Div. of Eng., National Research Council, Washington, D. C.

The investigation includes the effect on tractive resistance of the temperature of the differential of the vehicle, and of various types of tires. The gross tractive resistance is composed of several elements, such as engine, gearing, inelastic resistance of body, tire, windage, road surface, which distinguishes rolling, impact, and displacement resistance. Account must be taken of the absorption and giving out of energy in the rotating parts, in accelerating or retarding speed. The technique of these measurements is not settled. The measuring instrument may be an accelerometer used in coasting tests, or a dynamometer used in towing tests, each of which has its peculiar advantages. The first task is to determine the best instrument to measure the several elements into which the problem is analyzed. The Bureau of Standards has assembled apparatus which automatically records the measurements of about fourteen elements of car performance in road tests.

L. E. Conrad, M. Am. Soc. C. E., at Manhattan, Kans., is conducting a parallel investigation of the resistance of the air to the passage of vehicles.

At the University of Michigan, Professor Lay in a study of economic grades has towed four Packard trucks over gravel and concrete roads, after he had determined their characteristics in the laboratory. The gravel roads varied in condition from wind-swept hard surfaces to a loose gravel surface. The concrete road varied from a smooth finish to a somewhat wavy condition. The gasoline consumption was also reported. The trucks were 5 ton, with 3-ton pay load, and were operated at a standard engine speed of 1 000 rev. per min., the gears being shifted to meet road conditions. Professor Lay finds that, for these trucks, the tractive resistance, including rolling resistance, tire resistance, and windage, varied from 40 to 44 lb. per ton on the gravel roads, and from 22 to 26 lb. per ton on the concrete roads, all at a speed of 10 miles per hour.

At Ames, Iowa, Professor Agg has been investigating the economics of highway location and operation, especially with a view to determining economic grades.

The end in view is a sound economic theory of highway location and improvement. Each of these investigations forms part of a whole. The individual researches profit by intercommunication, and, in the end, the entire results are more likely to be useful in the formation of principles by a co-ordinating service.

In the field of design may be mentioned the experimental road at Pittsburg, Calif., and the Bates test road near Springfield, Ill., investigations of A. T. Goldbeck, Assoc. M. Am. Soc. C. E., of the U. S. Bureau of Roads, and the service test roads at Byberry and Lancaster, Pa., Casper, Wyo., Alexandria County, Virginia, Wilmington, Del., and Milwaukee, Wis. These tests are directed to the same end and will supply co-ordinated data through the Committee on Design; they will place the mechanics of slab design on a scientific basis.

In the field of highway transport, the pioneer transport surveys on the Connecticut roads, started by Charles J. Bennett, M. Am. Soc. C. E., State Highway Commissioner, have crystallized former vague discussions, and have stimulated further work. The examinations by A. N. Johnson, M. Am. Soc. C. E.,

of the University of Maryland, by N. W. Dougherty, M. Am. Soc. C. E., of the University of Tennessee, by Professor Agg in Iowa, by the Bureau of Public Roads in California, and less complete traffic counts in scores of instances—all these need to be reported and examined for fundamental laws of traffic by a Committee on Traffic Analysis.

A useful and money-saving activity is in the Research Information Service of the Advisory Board, by which a State is informed of the findings from investigations in another State. Cases can be cited in which one State has been saved the delay and cost of an investigation relating to the use of materials, when informed of the completed investigation available from the work of another State. The recent tabulation of the incomplete returns of the Highway Research Census of the Advisory Board indicates not only great activity in this field in the 430 projects, but the possibilities of economy in time and money and the avoidance of unnecessary duplication. Table 1 gives the number of individual research projects in highway engineering and highway transport.

TABLE 1.

	Economics.	Operation.	Design (road).	Construction.	Materials.	Total.
Colleges and universities	17	3	47	12	127	206
Industrial	1	1	3	0	19	24
Municipalities	5	1	6	1	18	31
State highway departments	13	2	25	8	101	149
State geologists	0	0	3	0	17	20
Total	36	7	84	21	282	430

It must be said, however, that duplication is not necessarily an evil; indeed, it is useful in fundamental matters, as in the case of the investigation of fatigue of concrete. The parallel investigations at Springfield, Ill., Purdue University, and the University of Maryland, with different methods of attack, will more certainly lead to the truth. However, in these cases, each will profit by communication with the other.

H. S. Mattimore, Assoc. M. Am. Soc. C. E., Engineer of Tests, State Highway Department, Harrisburg, Pa., is Chairman of the Committee of the Advisory Board on the Character and Use of Road Materials; he is also Chairman of the Committee of Tests and Investigations of the American Association of State Highway Officials. The latter Committee is divided into sub-committees, each representing a region of the United States. Mr. Mattimore is furnished by the Advisory Board with an administrative assistant, and this combined attack on problems relating to the materials of construction represents a well-organized committee activity.

These few instances will serve to demonstrate the value of the services of the Advisory Board in combining individual researches.

POLLUTION OF STREAMS BY PULP-MILL WASTES

By George C. Whipple,* M. Am. Soc. C. E.

The manufacture of wood pulp is a hydro-chemical process; consequently, pulp mills are always located on streams or canals. Large volumes of water are required and the waste waters which contain the spent chemicals and the washings of the pulp, pollute the streams. Whether this pollution does damage down stream depends on many factors, namely, the relative volumes of waste and of stream flow and the variations in flow; the quality of the river water and the character of the wastes; the opportunity for mixing, the velocity of the current, the existence of riffles, slack water, aqueous vegetation, re-aeration, sedimentation, and the many factors which bring about the natural purification of streams; and, finally, the use which is made of the river water down stream. These factors are common to all forms of stream pollution resulting from hydro-chemical processes, but there are no two places where the conditions are exactly alike. Every polluted stream has its own set of conditions, therefore, generalizations are hazardous. The speaker will only call attention to certain facts regarding pulp-mill wastes and discuss briefly the larger problem of stream pollution by industrial wastes.

It should be noted that the pollution of streams by pulp-mill wastes is only a part of a more general problem—the pollution of streams by trees and tree products—a problem which appears in many forms. Annually, the leaves fall, decay, and the products of their leaching are carried into ponds, brooks, and rivers. Trees die and gradually their wood decomposes. Trees are floated down stream to the mills, and their soakage tends to increase the color of river water, although the effect is not great. Logs are sawed; the saw-dust lies in heaps or washes into the rivers and the pollution takes a different form. All these processes are slow, and the resulting pollutions are regarded as natural. It is only when wood fiber is treated chemically, when vast quantities of logs are brought to one mill, that stream pollution becomes a problem. Pulp-mill pollution is also akin to that of strawboard waste pollution and to beet-sugar pollution, both of which involve the decomposition or chemical treatment of vegetable fiber and the use of water in large quantities.

There are three processes of making wood pulp, which need to be distinguished from each other because of the difference in their waste products. They may be referred to as (*a*) ground pulp; (*b*) soda pulp; and (*c*) sulphite pulp. The object of all these processes is to separate the fibrous from the non-fibrous parts of the wood, the fibrous materials being used for making paper.

Ground pulp is made by cutting logs into short lengths and pressing them against rough grinding wheels kept wet with water. The resulting crude pulp is washed, screened, deposited as a blanket, rolled, and dried. Mechanical pulp thus prepared contains not only cellulose but also lignin and resinous matter. The wash water contains a large quantity of wood fiber and extractive matter, sometimes 10% of the weight of the wood. Chemically, it resembles saw-dust, but is finely shredded instead of granulated.

* Cons. Engr.; Prof., San. Eng., Harvard Univ., Cambridge, Mass.

In the soda process, soft wood, usually poplar, is chipped and digested in hot caustic soda, the resulting pulp being washed, bleached, drained, and worked into the usual blanket form. Soda digestion yields a soft grayish-brown pulp and a black liquor which results from the decomposition of the lignin and resins. A large part of the soda used is recovered by concentrating this black liquor in evaporators, burning off the lignin and other organic materials in rotary furnaces, leaving carbon and sodium carbonate which is leached out and converted back to caustic soda by boiling with caustic lime. As a result, there is wasted a small volume of black-ash sludge which contains a little unleached soda ash, and a large volume of calcium carbonate sludge. A small volume of lime sludge resulting from the bleaching process is also wasted. The liquid wastes consist of wash waters from several parts of the process. They contain wood pulp, caustic soda, and hypochlorites, but are usually not large in volume. In recent years, electrolytic methods of producing caustic soda and calcium hypochlorite from brine have been introduced and savings made which have reduced the waste products. It is said, however, that for every ton of wood-pulp product about 150 lb. of caustic soda are lost.

If all the solid waste products from soda pulp mills were to be discharged into the streams, a serious pollution would occur. Sedimentation basins are usually provided and with them it is possible to retain the bulk of the sludge. The volume of the sludge is very large, however, and in the course of years large areas are required. Unless the sedimentation beds are well operated, foul liquids are likely to be discharged from them, either by drainage or by overflow at times of rain. On the whole, however, the soda process, with its recovery methods, destroys a large part of the lignins and wood extractives by burning, and the pollution of streams by soda pulp mills is more a mineral than an organic pollution.

In the sulphite process, the wood fiber is disintegrated by the use of the sulphites of calcium and magnesium with an excess of free sulphurous acid. Spruce and hemlock are the woods commonly used. The sulphite is manufactured by passing sulphurous acid through a suspension of milk of lime, or by burning sulphur and passing the fumes into an acid tower packed with crushed limestone or dolomite, water being allowed to trickle down as the fumes ascend. The sulphite solution runs out at the bottom of the tower and is pumped to the digesters in which the chipped wood is packed. After digestion for several hours under high pressure, the contents are delivered into a pit from which the waste liquor drains off, leaving the fiber. This waste sulphite liquor has the appearance of a thin molasses, and has a woody, acrid, sulphurous odor. It often contains as much as 10% of solid matter, consisting of wood fiber, lignin, resin, and wood extractives modified by the process. It generally contains sulphurous acid, sometimes as much as 0.5 to 1 per cent. The pulp is screened, washed, and perhaps bleached, and is worked into a blanket ready for paper-making, these processes yielding waste waters which contain fiber and other impurities. It will be observed that in the sulphite process the lignin, resin, and wood extractives are wasted to the stream, whereas, in the soda process, they are mostly burned. For this reason, the pollution of streams

by sulphite pulp mills has proved a more serious matter than that caused by pulp mills using the soda or the mechanical process.

The volume of the liquid waste from a sulphite pulp mill is large and may be as much as 800 to 1 000 gal. per ton of pulp. It is said that for each ton of finished pulp an equal quantity of organic material is discharged into the stream as waste liquor, together with about 400 lb. of sulphur in acid form or organic combination. The action of the bisulphites on the lignin is to produce a mixture of sulphonated substances which give rise to lignin-sulphonic acid or the calcium or magnesium salts of this organic acid.

Earle B. Phelps, Affiliate, Am. Soc. C. E., has given the following average analysis of two samples of sulphite liquor, which may be regarded as typical:

	Parts per million.
Total solids	95 000
Loss on ignition (organic)	82 000
Oxygen consumed	36 000
Ash (mineral)	13 000
Analysis of ash:	
Calcium (Ca)	2 200
Magnesium (Mg)	1 200
Sulphur (S)	2 000
Free SO_2	1 000
Total sulphur (mineral and organic)	10 500

Mr. Melville C. Whipple, who analyzed a sample of the waste sulphite liquor at one of the mills on the Black River, New York, found that it had an iodine absorption equivalent to 224 parts per million of SO_2.

As far as the speaker is informed, no successful method of utilizing or treating sulphite wastes has yet been devised. Its acid condition precludes the use of bacterial methods without prior neutralization and its large quantity of soluble matter makes settling processes of little avail. Evaporation, which is an expensive process, seems to be a necessary preliminary to any method of utilization. The use of detention tanks for equalizing the discharge instead of allowing heavy discharges to occur at intervals, is of some benefit but does not solve the problem. If one considers the damage done and also the value of the substances wasted, it is evident that the prevention of pollution by pulp-mill wastes is worthy of careful study by chemical engineers. Sutermeister states that, in Europe, mills have been obliged to close, because it has been impossible to purify the waste liquors sufficiently to comply with legal requirements. Attempts have been made to use the sulphite liquor as a road binder, a binder for moulding sands, a binder for powdered coal, a fertilizer, a sizing for paper, a food for cattle, a source of dye-stuffs, a material for insulation and artificial leather, or a source of alcohol. The speaker has no practical suggestions to make at this time on the utilization and treatment of wastes but, instead, will discuss briefly the effect of sulphite wastes on streams.

The peculiar character of waste sulphite liquor is its sulphur content, its acidity, and its high organic content. Sulphite mills and often the villages in which they are located, reek with the characteristic odor. Sulphurous acid

escapes into the air in such quantities that the water supply, used for process purposes, when exposed to the air, absorbs it, the natural alkalinity of the water being reduced by measurable amounts. Sulphur in other forms may also be absorbed by the process water, partly from the air, but even more from the pulp during the washing processes. One consequence of this is, that the troughs and channels and even the screens and wash-boxes in a pulp mill become coated with slime which is shown by the microscope to consist of *Beggiotoa, Sphærotilus, Leptomitus,* and other fungi which require sulphur for their growth. The slime problem is often serious in pulp mills and frequent shut-downs for cleaning purposes are required. The slime becomes dislodged from the troughs and settles on the pulp blankets, making dirty spots which affect the sale of the product and sometimes require the pulp to be re-washed. A few years ago, at one of the Canadian pulp mills, the speaker experimented on the control of slime growths by the use of copper sulphate coupled with other chemical treatments, but this experimental work had to be given up on account of the World War. Sufficient was accomplished, however, to warrant the statement that the slime growth can be controlled by proper treatment.

It is a fact which is more than a mere coincidence, that these three filamentous fungi, *Sphærotilus, Beggiotoa,* and *Leptomitus,* found in the water of pulp mills, are also found in sewage polluted waters. Together with *Thiotrix* and, perhaps, *Saprolegnia,* they appear to require sulphur and, either by choice or effect, are found in waters of low dissolved oxygen content. These fungi are often found on sticks and stones in the streams below sulphite mills, appearing as clumps or tufts of fibers, sometimes white like cotton, but more often cream-colored, gray, or salmon pink. When they decay, they blacken and become foul smelling, a condition usually associated with septicity, that is, with lack of oxygen and bacterial decomposition.

Deposits of pulp sometimes occur below sulphite mills where there is slack water. The shores often become covered not only with the fungi mentioned, but with growths of *Oscillaria,* the long bluish-green filaments of which enmesh the pulp. In the sunlight, bubbles of oxygen are generated, which also become enmeshed and float great masses to the surface of the water. Various animal forms take shelter in these oxygenated masses, and other algæ grow on them until they become great centers of microscopic life. If there are other pulp or paper mills down stream, the quality of the water at the lower mills is likely to be seriously impaired for washing purposes. These growths can be kept down by the proper application of copper sulphate.

The practical effect of discharging sulphite wastes into rivers depends on the various factors already mentioned. One effect is to make the stream unsightly by reason of deposits of pulp and fungi and the algæ growths previously mentioned, and if a stream once becomes foul in this way, it tends to become a catch-all for many kinds of débris and filth. Uncleanliness is contagious. M. O. Leighton, M. Am. Soc. C. E.,* described the effect of the pollution of Ausable River with sulphite wastes for a distance of twenty miles, and even

* "On the Pollution of Lake Champlain", *Water Supply and Irrigation Paper No. 121,* U. S. Geological Survey, 1905.

into Lake Champlain. He also described the pollution of Bouquet River with soda pulp wastes. Some thought at one time that it increased the algæ growths in the lake, but this was never definitely proved. Phelps* has also written on the general subject of sulphite pulp wastes and has described processes and proposed methods of treatment.

Pulp-waste pollution tends to drive away fish. The fine pulp gets into their gills and the sulphite pollution may kill them by exhausting the dissolved oxygen. The New York Conservation Commission has recently assembled data showing the tolerance of certain fish to industrial wastes and among them are the following: Bass, perch, and trout were killed in about 1 day by sulphite wastes diluted 10 to 1 and when diluted 75 or 100 to 1 trout fry and bass were killed in several days. Dilutions of 100, 200, and 500 to 1 did not kill perch in several days. That death was due to suffocation rather than poisoning was indicated by the fact that a 50 to 1 dilution of the sulphite liquor artificially aerated did not kill perch or bass in 29 days. Bleach sludge diluted 500 to 1 failed to kill perch in 14 days, and aerated bleach sludge diluted 400 to 1 did not kill perch in 5 days. Even if the dilutions are far greater than these figures, sulphite wastes may tend to drive away certain kinds of fish which, like trout and salmon, apparently seek pure waters. There can be no doubt that there are selective differences among fish as to oxygen requirements. It is said that certain of the grosser fish are capable of tolerating pulp-mill wastes, but acquire disagreeable tastes which make them nonedible.

Sulphite wastes may also injure the water of a stream in its use as a boiler water. In at least one instance, such wastes have been known to make the water of a large stream acid, and the damage done by them is often more subtle than that produced by the gross pollution in the vicinity of the mill. The salts of lignin-sulphonic acid, or whatever the compounds may be which result from the action of the acid bisulphite on the lignous and resinous compounds of the wood, have a most persistent odor; even when the sulphite wastes are greatly diluted, it can still be recognized. Formerly, the water supply of Augusta, Me.,† was taken from the Kennebec River, and the imperfectly filtered water often had a noticeable taste and odor resulting from the pulp-mill wastes at Winslow, 17 miles up stream. The water supply of Bangor, Me., is taken from the Penobscot River, and before filtration has the same characteristic taste. At times, the alkalinity is materially reduced and the coagulation of the water rendered difficult. At Watertown, N. Y., the Black River, polluted by sulphite wastes at Deferiet, Herrings, Carthage, and Lyons Falls, has the same taste and odor, which the filter has never been able wholly to remove. The odor is not removed by aeration and the use of chlorine seems at times to intensify it. The municipal authorities of Augusta wisely abandoned the Kennebec River supply and substituted a supply from a pond not far away. At Bangor, in spite of advice to seek another source, the Penobscot supply has been retained and improvements made in the filter plant.

* "The Pollution of Streams by Sulphite Pulp Wastes", *Water Supply and Irrigation Paper No. 226*, U. S. Geological Survey, 1909.

† "Quality of the Kennebec River Water", *Water Supply and Irrigation Paper No. 198*, U. S. Geological Survey, 1907.

Unfortunately, it is not the odor alone which causes trouble with down-stream water supplies. The sulpho-lignous organic matter interferes with the coagulation of the water, reducing the efficiency and increasing the cost of filtration. The exact nature of the reactions has not yet been accurately determined, but some studies have been made at the Watertown filter, which have led to modifications in the process and improvements in the results.

The water of the Black River is deeply stained, its average color being about 80 and its maximum color in different years running up to 135. The quantity of carbonaceous organic matter in the water is greater than that represented by the color. In a discussion* by the speaker before the Society in 1901, on decolorization of water, it was shown that in stained waters the organic matter, as indicated by the "oxygen consumed" test, was 0.125 part per million for each unit of color; but, at Watertown, tests have shown that the oxygen consumed corresponding to a unit of color is often double this quantity. The oxygen consumed is also higher at seasons of low stream flow, when the dilution is less than at other times. Some of the colorless organic matter found at Watertown may be due to sewage pollution, but inasmuch as sulphite waste shows a high figure for oxygen consumed, it is reasonable to suppose that these wastes are an important factor in the Black River. There are visible evidences of this pollution at the Watertown filter, as particles of woody fiber from the waste pulp can be easily detected by the eye; in fact, the superintendent of the filter is in the habit of making a daily record of the approximate quantity of pulp observed in the raw water. Phelps has shown that the "oxygen consumed" in sulphite waste liquor is from 36 000 to 60 000 parts per million; therefore, even after being diluted several thousand times, the water would contain as much organic matter, measured in terms of "oxygen consumed", as would correspond to a color of 100 units. It has been found, as a result of filtration experience, that the quantities of alum required for the coagulation of unstored colored waters free from sewage and industrial wastes, ordinarily bear a fairly definite relation to the color of the water; but in the case of the Black River water, the quantity of alum required is relatively great, largely because of the organic matter from the sulphite wastes. When the water filters were designed by Allen Hazen, M. Am. Soc. C. E., in 1903, it was decided, as a result of experiments made with colored waters, that a period of coagulation of 4 hours would be required, a time considerably longer than previous practice. Experience, however, has proved that even this was not long enough. In 1917, an existing settling basin was converted into a coagulation basin which increased the period of coagulation to about 8 days, and the results of this longer period have proved to be beneficial. Better results have been obtained, although the quantity of alum used has not been lessened. It will be seen, therefore, that, in various ways, sulphite waste pollution increases the cost of water purification.

This is not the whole story, however. Unless coagulation is well accomplished, that is, unless enough alum is used and the time of coagulation is sufficiently long, some of the alum in combination with the organic matter will pass through the filter. Three objectionable results follow, namely, the

* *Transactions*, Am. Soc. C. E., Vol. XLVI (1901), p. 141.

filtered water retains some of the "pulp-mill taste"; the water when heated for ordinary domestic service becomes corrosive, attacks iron, and causes red-water troubles; and the bacterial removal by filtration is not complete. These are all serious matters. Even with what is regarded as good coagulation, the sulphite taste may persist in filtered water.

At one time, as a result of experiments at Watertown, it was thought that basic alum was generally preferable to acid alum for water-filtration purposes. Later studies have made this doubtful. There are times, perhaps, when an acid alum may give better results than a basic alum, waters differ in this respect. With alum made at the filtration plant, as is now often the case, it may be possible to modify its character so as to obtain the best results for any particular water.

Engineers once thought that they had obtained a fairly complete understanding of this problem of coagulation, but experience with mechanical filters, although on the whole successful, has shown that there are many things to learn about coagulation, especially the coagulation of soft, colored waters and waters polluted with such wastes as those from pulp mills. The nature of the colloidal substances, the possible protective effect of colloids on bacteria, the relations between chlorine and organic matter, and the reaction between sulphate of alumina and organic colloids, are some of the subjects which chemists and biologists now have under consideration.

Enough has been said, perhaps, to show that the pollution of streams by pulp-mill wastes is a matter which may cause damage to down-stream riparian owners and affect the health and comfort of many people in communities which take their public water supply from polluted rivers. It also has a bearing on the general problem of conservation of natural resources. Pulp mills not only spoil the rivers, but they despoil the forests. Wood pulp is a useful substance, but there is hardly any raw material that is wasted so lavishly. Without doubt, chemical engineers can solve the pulp-mill pollution problem if capitalists will provide the means, the laws are ample to protect the down-stream riparian owners if they are enforced but, in the speaker's opinion, a better solution of the problem lies in preventive measures, that is, in measures which will protect the forests. Conservation demands that tree trunks suitable for lumber be used for that purpose. It is necessary to find methods of obtaining cellulose such that the newspaper which is in use a day and the paper towel which serves the purpose of a moment are not made at the expense of a tree which has taken twenty years to grow.

The speaker may be wrong, but it appears to him that there is now a greater indifference to stream pollution, a greater laxity in enforcing laws, than was the case before the World War. Probably, this is merely a part of the widespread lapse from law and order unfortunately so prevalent at this time. It is a condition which must not be allowed to continue. The pendulum may swing, perhaps, from too much laxity to too much rigidity. The present is a good time, therefore, to consider carefully in a broad spirit of co-operation and in a scientific manner, the best way of dealing with the whole subject of stream pollution. Certain things have been demonstrated by experience. The pollution of a stream may steadily increase without serious offense and with-

out attracting much attention, until suddenly a crisis is reached, the dissolved oxygen disappears from the water and the river is wrecked. It has been learned that it is cheaper and safer to protect a down-stream water supply by filtration than by up-stream treatment of sewage, but that a single line of defense is not enough. It has been learned that many kinds of manufacturing wastes impart tastes and odors to waters, which cannot be readily removed by water filtration and that chlorination sometimes accentuates them, and that these tastes and odors may be produced even when the polluting substances are very dilute. It has been learned that river-pollution problems are not all related to the public health. It has been learned that because rivers flow between States and from one State into another, the control of pollution by independent State action is not adequate to meet the situation, and yet the police power resides in the sovereign States. It is evident that some new mode of procedure is necessary to deal with the situation; yet it is doubtful whether any better legal remedies will ever be devised than the application of the principles of common law and the exercise of police power by the separate States. The speaker does not favor delegation of the police power to the Federal Government in this matter, and, at present, the powers of the Federal Government are limited to navigable streams.

A uniform policy throughout the country is much needed. To secure this, a thorough scientific, economic, and legal study of the whole subject of stream pollution should be made on a National scale by a joint committee representing the National bodies interested in the different phases of the subject, under the sponsorship of one of the Federal Departments with Congressional authority and an adequate appropriation. This committee, after several years of investigation, should formulate a policy, suggest model laws, and recommend their adoption by the several States. It may be possible to organize river districts independent of State boundaries, with commissioners whose duties would be to secure co-operative efforts to keep the rivers clean and to utilize the waters of the streams to the best advantage of all the people living within the districts, leaving all legal actions where they now are and where, in the interest of good government, they should remain. The whole subject at present is in a chaotic state. Some State Departments of Health attempt to exercise almost despotic powers over the streams; in other States, their action is chiefly advisory; and, in some States, there are pollution commissions independent of the health authorities. Although there has been a recent laxity in law enforcement, it is hoped that the next move will not be toward a too arbitrary enforcement of ill-considered laws, but that means will be found to accomplish by co-operative effort what is so difficult to accomplish by law. It would appear that the Society is one of the organizations which should take the lead in this important matter of conservation.

WASTES FROM PULP AND PAPER MILLS CHEMICALLY CONSIDERED

By H. W. Clark,* Esq.

There are two main points of view concerning pulp and paper-mill wastes: (1) That of the owner of the mill who wishes to discharge wastes of no value into the streams on which the mill is located; and (2) that of the sanitary engineer who wishes to keep the stream in as unpolluted a condition as can be reasonably required.

Where industrial streams are not used for water supply, about all that is necessary in the treatment of industrial wastes is purification sufficient to prevent the stream from becoming a nuisance to the community, on account of its appearance, deposits on its banks, and odors. If possible, it should be kept clean enough to be a source of enjoyment to this community. Where streams are polluted by pulp and paper-mill wastes too large in volume to be properly diluted and cared for by the volume of water into which they go, and this stream must be used farther down on its course as a water supply after filtration, efficient treatment of these wastes is not only desirable, but necessary.

The speaker began to investigate the pollution of streams by wastes from pulp and paper mills about twenty-seven years ago and has been obliged to consider the subject of paper-mill wastes both from the viewpoint of the sanitary engineer and the manufacturer who hesitates to expend large sums to satisfy extravagant demands for the prevention of stream pollution.

Many kinds of wastes result from the making of paper and a more or less complete classification of them is as follows: (1) The wastes from pulp mills where wood, either poplar or spruce, is treated; (2) the wastes from digesters in mills making paper from material other than wood pulp, that is, from old rope, carpets, bagging, old paper, etc., and as this stock is treated in digesters, the wastes resemble those from wood-pulp mills; (3) the waste liquor, very polluted, produced by washing and beating such stock before it is treated in digesters and made into paper; (4) the waste from mills in which rags only are the stock treated with milk of lime in rotary boilers; and (5) the wastes from making pulp of all kinds into paper.

At present, the wood-pulp mills treat either poplar, spruce, or hemlock, the poplar being treated by the soda process and the spruce and hemlock by the sulphite process. Owing to the decrease in the supply of poplar, the sulphite process is perhaps gaining materially over the soda process. Two classes of waste liquor are produced, one from each process. At many mills, the digester liquor from the soda process is evaporated and the used chemicals recovered, thus preventing the discharge of much waste liquor. The liquors from the sulphite process generally go to waste, that is, are discharged into the stream on which the mill is located. The following statements concerning two typical pulp mills in Massachusetts, one using the soda process and the other the sulphite process, can be made.

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In the pulp mill using the soda process, 2 500 lb. of soda ash are used for each charge in the digester, this soda being causticized 90% before use and the residual calcium carbonate resulting from this process used in sizing. Of the 2 500 lb. of soda used in each charge, about 500 lb. is new, whereas the remainder has been recovered from the black ash of previous digestions, the digester liquor having been evaporated and calcined. At this mill, about 5 000 000 gal. of this digester liquor is produced annually, of which only about 10% is wasted, being lost during the washing of pulp after digestion, the remainder being evaporated for this recovery of chemicals. The analysis of this waste liquor from the soda process is as follows:

	Parts per million.
Total solids (no suspended solids present)	132 000
Loss on ignition	72 100
Fixed solids	59 900
Alkalinity (methyl orange)	39 000
Free ammonia	34.00
Albuminoid ammonia	72.00
Oxygen consumed	27 600

Volume for volume, this liquor is perhaps one hundred times as deleterious to a stream as domestic sewage. It contains a large quantity of organic matter and its oxygen demand while comparatively slow, is enormous, perhaps twenty times that of average domestic sewage. In 24 hours, 1 gal. of this waste will absorb 10.78 grammes, or 7 544 cu. cm., of oxygen. It is so alkaline—practically 4%—that it destroys bacterial life in the water into which it flows, and it is deadly to fish. Fortunately, the volume wasted is small and will probably decrease as the recovery of chemicals increases in favor.

A different story must be told of the liquor from the sulphite process. At the plant under discussion none of the digester liquor is recovered, about 17 000 000 gal. being discharged annually into the stream on which the mill is located.

The speaker believes that in a few cases this liquor is concentrated by evaporation or is treated so that it becomes useful for certain purposes. It contains 4 000 parts per million of organic acid, which should make its recovery of value. During the World War, experiments were made looking to the recovery or manufacture of alcohol from it. An analysis of this waste from the sulphite process is as follows:

	Parts per million.
Total solids (no suspended solids present)	74 100
Loss on ignition	66 300
Fixed solids	7 800
Alkalinity (methyl orange)	—800
" (phenolphthalein)	—5 000
Sulphur dioxide (as sulphurous acid or sulphites)	400
Free ammonia	146
Albuminoid ammonia	62
Oxygen consumed	34 600

This waste is acid and contains practically as much organic matter as the soda-process waste. Its oxygen demand on the stream it enters, appears to be somewhat less, and it is slightly less deadly to bacteria and to fish. In 24 hours, 1 gal. of this waste will absorb 8.71 grammes, or 6 088 cu. cm., of oxygen. There is no reason why the treatment of this liquor should not be demanded where sanitary conditions require it, as it probably can be evaporated and products recovered which would pay at least for the cost of treatment. Neither of these liquors can be satisfactorily treated by any known method of purification other than evaporation and recovery of bodies of value. They are exceedingly inimical to bacterial life, and if mixed with sewage, or other liquors, and passed to filters, they destroy the value of such filters.

As stated previously, wastes from certain paper mills consist of the liquors from washing old stock worked up, from treating this stock in digesters, and from the paper machines. In such mills, soda ash, lime, bleaching powder, white clay, etc., are used and a large part of these chemicals are wasted. There is generally no saving of soda ash or other chemicals in mills using mixed stock. The true wastes, however, are largely carbonaceous, and in order to purify or clarify them satisfactorily, dependence must be had on straining, sedimentation with or without the use of precipitants, and filtration through filters of sand, cinders, or like material. In the treatment of domestic sewage, nitrification by bacterial action is depended on where a high degree of purification is required, but the small amount of nitrogen in paper-mill waste renders this process of little value.

Investigations at the Lawrence Experiment Station showed that when a liquor to be treated contained an x amount of nitrogen and from $10\,x$ to $12\,x$ of carbon, nitrification would still take place in a filter, but that it was practically killed when the amount of carbon was $13\,x$ or $14\,x$, or greater. In this investigation, studies of the behavior of paper-mill wastes under certain conditions of filtration were made, together with complete analyses of such wastes, to determine the relative amount of nitrogen and carbon present in them. Some bacterial action, however, does take place in filters receiving carbonaceous wastes, this purification without nitrification being due to certain bacteria which are active under conditions that prevent nitrification, but which cause chemical actions that break down organic matter and result in the setting free of carbon dioxide and nitrogen.

Much of the organic pollution in paper-mill wastes is in suspension, and a considerable part is quite readily removed by passing the liquors through fine wire screens. In some mills, a large part of the pulp formerly wasted from the paper machine is saved in this way, and a large part of the dirt in various wash waters can also be screened out. Besides screening, sedimentation is depended on at many mills to save pulp, and at such places aluminum sulphate is also generally used to aid sedimentation. The paper fiber saved in this way much more than repays the cost of construction and operation of sedimentation tanks and the aluminum sulphate used. The aluminum hydrate produced and unavoidably mixed with the pulp used, is of value, generally speaking, in the weighting of paper and often as much as 4 or 5 grains per gallon of sulphate is

used in this saving of fiber. The quantity of oxygen absorbed in 24 hours by wastes from a mill representing the class using mixed stock is shown by Table 2.

TABLE 2.

Sample.	Dissolved Oxygen Absorbed by 1 Gal. in 24 Hours.	
	Grammes.	Cubic centimeters.
Rotary boiler wastes	2.2720	1 589.0
Washer wastes	0.4920	344.2
Settled washer wastes	0.2272	158.9
Machine wastes	0.1136	79.5
Settled machine wastes	0.0304	21.2

When the speaker first began to study paper mills and their wastes, hardly a "save-all" was in use and none of the elaborate sedimentation and precipitation tanks that are in common use to-day. Besides the saving of machine wastes by the tanks, etc., the pollution of streams is prevented by screens and sedimentation tanks through which are passed the waste liquors from the washing of old stock. About 30% of the matters in suspension is generally eliminated in this way by 4 hours of sedimentation and from 90% to practically all the matter in suspension can be removed if 75 to 100 grains of aluminum sulphate per gallon, are used. This means from 10 000 to 14 000 lb. per 1 000 000 gal. and is a large amount of precipitant, but it is being used in some mills where it is necessary to prevent the pollution of streams. Where several million gallons of waste are produced by one mill, the removal by sedimentation alone of even 30% of the matter in suspension means much reduction of river pollution. More than this, sedimentation and filtration can be depended on to treat the mixed wastes for further purification. The rates of filter operation vary with the quality of the liquor being treated, but rates of from 200 000 to 500 000 gal. per acre per day are used with good results. By a combination of all these methods—screening, sedimentation, and filtration—it is often possible to remove 70% or more of the primary polluting matters of paper-mill wastes. Apparently, however, the only wastes recovered with profit as yet are those from the soda-pulp process and the wastes from the paper machine. Much research work is being done, and it is probable that valuable discoveries will be made.

THE TREATMENT AND DISPOSAL OF STRAWBOARD WASTES

By H. B. Hommon,* Esq.

This paper gives a complete account of the studies made at the American Strawboard Mill, Noblesville, Ind., for developing methods to treat wastes resulting from the manufacture of paper from straw.†

Process of Making Paper from Straw.—Strawboard mills are generally located in wheat-growing districts. The straw is purchased from the farmers, baled, and shipped to the mills where it is stacked in large piles. The first step in the process of making paper is to cook the straw in large ellipsoidal rotating steel boilers with steam and lime, about 1 ton of the mixture to 6 tons of straw, until the woody fiber is softened, and the whole mass has been reduced to a dark yellow pulpy consistency. The yield of pulp at this point is about 75 to 80% of the weight of the original stock. The pulp is removed from the boilers, allowed to drain, and then placed in beaters and washers, where it is broken up and the lime, fine particles of straw, and other foreign material washed out.

From the washing machines, the pulp is carried by water to the paper machines, where it is taken up on felts and delivered to a train of hot rolls that turn out the finished product.

Volume of Wastes Produced.—Of the many sources of wastes from a strawboard mill, only two produce large quantities, namely, the washing and the paper machines. These two wastes come from separate parts of the mill and differ widely in composition, but they are approximately the same in quantity. While the tests were being conducted at Noblesville, a survey was made of seven other strawboard mills, in order to determine how far the analyses and flow data obtained at the experiment station were representative of the entire strawboard industry.

As already stated, the pulp from the cookers or rotaries is washed to remove the lime, fine particles of strawboard, and other foreign material. The quantity of water used for washing the pulp varied considerably at the different mills where measurements were made, but the average was 18 200 gal. per ton of strawboard made. This figure checks closely the average for a long period at the mill where the experiments were conducted.

The waste from the paper machines consists of the water used to carry the pulp to the train of rolls. The measurements obtained at the various mills visited, showed variations, but the average, 19 900 gal. per ton of finished product, was almost identical with that obtained from the flow data at the experiment station. Therefore, about 38 500 gal. of waste was produced for each ton of strawboard made.

Composition of Wastes.—The waste from the beaters or washing machines was by far the more concentrated of the two, as it contained the chaff, minute particles of straw, and small particles of other organic matter washed from

* San. Engr., U. S. Public Health Service, San Francisco, Calif.

† Data taken from *Bulletin No. 97*, U. S. Public Health Service.

the pulp after digestion in the boilers. This waste also contained the lime washed from the digested pulp.

When the survey was made of the seven strawboard mills to determine the volume of wastes produced, a series of composite samples were collected from the wastes from the washing and paper machines and shipped to the central laboratory in Cincinnati, Ohio, for analysis. The results of the analyses of the washing machine wastes showed that the total suspended matter varied from 3 000 to almost 9 000 parts per million; the suspended matter in the supernatant liquor, after 24 hours of settling, varied from 450 to 2 400 parts per million; the organic nitrogen, from 47 to 105 parts per million; the alkalinity as carbonates, computed as $CaCo_3$, amounted to 1 870 parts per million; the oxygen consumed varied from 1 480 to 3 400 parts per million; and the oxygen demand, 10 days at 20° cent., from 1 850 to 3 600 parts per million.

The waste from the paper machines contained only the small quantity of fiber and other finely divided material that passed through the screens, together with the lime left from the washing process. The analyses from the mill where the tests were made and from the other seven mills covered in the survey, varied within the following limits: Total suspended matter, range 440 to 991 parts per million; suspended matter in supernatant liquor after 24 hours of settling, 60 to 460 parts per million; total organic nitrogen, 10 to 43 parts per million; carbonate alkalinity found in only three wastes of the eight reported; oxygen consumed, 191 to 970 parts per million; and oxygen demand, 10 days at 20° cent., 111 to 890 parts per million.

The analyses of the composites of the two wastes at the experiment station for the period of the tests checked the averages of the results from the mixed wastes at the other seven mills, within reasonable limits.

On the basis of the figures given for the average total volume of waste per ton of strawboard manufactured, and on the rated capacity of the mills in the United States in 1918*, there were approximately 20 000 000 000 gal. of waste discharged from the various plants during the year. Computed on the basis of the average suspended matter in the mixed wastes from all the plants visited, there were approximately 161 000 tons of dry suspended solids discharged from the strawboard mills in 1918. Much of the material was short fibers of straw, which the type of machinery in use at that time could not save.

Treatment of Wastes.—The experiment station was operated at the American Strawboard Mill from January 1, 1915 to June 30, 1916. The test units consisted of two settling tanks each 8 ft. in diameter by 10 ft. deep; one chemical reaction tank, 6 ft. in diameter and 10 ft. deep; one chemical settling tank, 8 ft. in diameter and 10 ft. deep; one chemical solution tank, 5 ft. in diameter and 6 ft. deep; one mechanical filter, 8 ft. in diameter and 6 ft. deep; two filters, 8 ft. in diameter and 10 ft. deep; one filter, 6 ft. in diameter and 10 ft. deep; and one filter, 8 ft. in diameter and 6 ft. deep.

From January 1 to July 1, 1915, the plant was operated on the principle of mechanical filtration. Alum alone and lime and iron were used as coagulants. The results obtained were unsatisfactory, due mainly to the large

* Lockwood's "Directory of the Paper and Stationery Trades", 1918.

quantities of chemicals required for efficient coagulation, the very low rates of filtration through the mechanical filter, and the high percentage of water required to wash the filter. As this method of treating the waste was impractical, further description of the plant, or discussion of the analyses of the various effluents, will be omitted for this period of operation.

During July, 1915, the plant was remodeled and operated until the close of the tests on the principle of biological treatment of the wastes. The two settling tanks were retained and operated as during the first period, but all the other tanks were turned into filters. The filters were constructed as follows: One, 6 ft. in diameter, having 5 ft. of limestone screenings; one, 8 ft. in diameter, having 5½ ft. of 1½-in. limestone; one, 8 ft. in diameter, with 5 ft. of unscreened cinders; one, 8 ft. in diameter, with 5 ft. of screened cinders; and one small screened cinder filter, 4½ ft. deep.

Results Obtained at the Experiment Station During Period 2.—The test units were operated for about one year in this period, and the results showed that strawboard wastes of the character produced at the American Strawboard Mill in making paper from straw could be treated so that the final effluent could be discharged into a stream of water having a dilution factor of at least 5 or 6, without creating a nuisance in the stream, or killing fish.

The weighted average for the suspended matter in the mixed raw waste for Periods 1 and 2 was 1 760 parts per million. During Period 2, the averages for total nitrogen and oxygen consumed were 29 and 910 parts per million, respectively. The carbonates and bi-carbonates were 141 and 598 parts per million, respectively. The oxygen demand at room temperature for 24 hours was 40 parts per million.

The raw waste was settled in tanks for about 2 hours, and the average suspended matter in the effluent was 680 parts per million, which is a reduction of 53 per cent. This was computed on the suspended matter in the raw waste during Period 2. The advantage of longer storage periods than 2 hours was practically negligible. The total nitrogen averaged about the same for the tank effluent as for the raw waste. The carbonate alkalinity was reduced by the tanks from 119 parts per million in raw waste to 63, and the bi-carbonate was increased from 533 to 558 parts per million. The average for the oxygen consumed was reduced from 720 to 575 parts per million.

The effluent from the tanks was treated on filters of the different sizes, kinds, and depths of filtering material already described. Filters of coarse and fine limestone and of unscreened cinders did not give satisfactory results and discussions of them will be omitted. Two filters of screened cinders, one 8 ft. in diameter and 5 ft. deep, and the other, 4.5 sq. ft. in area and 4.5 ft. deep, produced satisfactory effluents. The smaller of the two filters was in operation 11 months and the larger, 9 months.

The average analyses of the effluent of the large, screened, cinder filter compared with the average results from the influent are given in the Table 3.

The suspended matter in the effluent of the large, screened, cinder filter had none of the characteristics of that in the effluent of the settling tanks or in the raw waste. It was more like humus. The final effluent had a slight

yellow cast and there was no odor. The nitrification was low, but samples diluted 1:3 with tap-water were stable by the methylene blue test for 90 hours.

TABLE 3.

	Influent, in parts per million.	Effluent, in parts per million.
Suspended matter	918	115
Carbonates as $CaCO_3$	121	29
Bi-carbonates as $CaCO_3$	528	417
Oxygen consumed	742	133
Oxygen demand (24 hours, room temperature)	54	9.5
Nitrogen, total organic	29	9.1
Nitrogen as nitrites	....	0.035
Nitrogen as nitrates	....	1.3
Hours to reduce methylene blue at room temperature, dilutions, 1:10 and 1:3, respectively	40	90

In considering the results of the analyses of this filter effluent, it should be borne in mind that the organic matter of the waste was mostly cellulose that had been worked over in the bed, and that in its final condition it was not different from the material washed from woods or lands covered with decayed vegetation. When the filter was examined at the close of the tests, the top layer of material had a deposit resembling the original pulp; but below the upper half of the bed, the color of the deposit was dark and it was alive with earth worms and other small organisms. The odor was much the same as that noted when digging into soil rich with decayed organic matter.

The filter contained considerable organic matter, but it did not have the same effect on the effluent that an equal amount would have on a sewage filter, probably for the reason that the clogging material was of uniform composition and the life in the filter was capable of reducing it before anærobic action could be established. After the filter was examined, it was flushed with water and the filtering material thrown out of the beds. The first water from the flushing carried out large quantities of black flocculent matter and masses of earth worms and other organisms, and, later, the accumulation from the top layers came through. While digging out the cinders, it was observed that the voids were fairly clear and that the material remaining after flushing was gathered around the filter media. It is believed that cinders screened over a ½-in. screen, or thoroughly washed with water, can be used for many years to treat settled strawboard wastes, but it may be necessary to flush the beds about once a year to remove the clogging material from the lower half of the bed. There probably would be no objections to using settled waste for flushing, but if water is available it should be used, as it would freshen the filter and not add the same burden of organic matter to the receiving body of water that would result from using the settled waste.

Settling Period and Rates on Filters.—The settling periods in the tanks at the experiment station varied from 2 to 8 hours. The suspended-matter results for the various periods showed conclusively that 2 hours of settling was sufficient to remove practically all the settleable matter that could be removed by tank treatment in an economical period of time. The screened

cinder filters were operated for several months at a net rate of 400 000 gal. per acre per day, and the results obtained were satisfactory. This rate has been set as the maximum for settled strawboard wastes, and it is believed that it can be maintained for many years, provided the filters are flushed with water or tank effluent whenever they become clogged so that the influent does not flow readily through the beds.

Treatment of the Wastes by the Activated Sludge Process.—Laboratory tests were made to determine whether this process would be applicable to this class of wastes. Air was blown through sewage in large-bore, glass tubing until a satisfactory floc had been obtained and then waste was added gradually to replace the sewage. The results obtained after prolonged aeration were not satisfactory as the supernatant liquor after settling 30 min. invariably contained about 105 parts per million of suspended matter and the color was only slightly less than that of the raw waste. The analyses did not show as high degree of purification as that obtained by the tanks and filters.

Accumulation, Composition, and Disposal of Sludge.—Careful measurements and analyses were made of the sludge deposited in the tanks. The measurements showed that about 2.75 tons of dry solids were deposited per 1 000 000 gal. of waste treated and this, computed to 87% wet sludge, gives 24 cu. yd., or 0.9 cu. yd. per ton of product.

The analyses of the sludge in different stages of decomposition, and of horse manure, are given in Table 4.

TABLE 4.—Analyses of Strawboard Waste Sludge and Horse Manure.

Source.	Percentage (Dry Basis 100° Cent.)			
	Total nitrogen.	Calcium as $CaCO_3$.	Total phosphorus as P_2O_5.	Total potassium as K_2O.
Fresh sludge from settling tank.....	0.69	21	0.38	0.54
Digested sludge from settling tank....	1.0	23	0.48	0.42
Sludge from old settling pond.....	0.66	37	0.29	0.25
Horse manure* (including litter.).....	1.7	....	0.26	1.50

* Data from *Bulletin No. 246*, Ohio Agricultural Station, p. 726.

The disposal of the sludge presents a rather difficult and expensive problem. On the basis of the data obtained in the tests, there would be produced at the American Strawboard Mill about 3 tons of dry solids per 24 hours, or about 26 cu. yd. of wet sludge. The only practical method developed for disposing of the sludge was to dry it on shallow beds of cinders and apply it to land under cultivation for grain grown in sections where wheat is raised. Field tests were made, using the dried sludge as a fertilizer, and satisfactory increases in yield were found for corn when 8 tons per acre were used. As nearly as could be determined, the sludge had about the same value as a fertilizer as ordinary yard manure. In small box tests, the dried sludge invariably increased the yield of wheat, oats, and vegetables.

Effect of Filtered Waste on Fish Life.—A small box, 3½ ft. long by 2½ ft. wide by 1 ft. deep, was placed under the outlet from the large, screened filter bed. The effluent discharged into this box was diluted with 1 part of fresh water, and into this mixture 20 minnows were placed. In this dilution, the fish lived from May 8th to July 1st, when the experiment station was dismantled.

Treatment Plant Recommended.—On the basis of the data obtained in the tests, it was recommended in the report that settling and sludge storage capacity of about 975 cu. ft. per ton of daily output from the strawboard mill be provided in ordinary plain settling tanks. For drying the sludge, it was recommended that 875 sq. ft. of filter area be provided for every ton of strawboard made and that the beds be made of unscreened cinders about 10 in. deep.

The filters used in the tests that were most satisfactory, were made of screened cinders 5 ft. deep, and the highest rate of treatment that gave satisfactory results was 400 000 gal. per acre per day. On the basis of these data, it was recommended that 4 171 sq. ft. of filter area be provided for each ton of product made, or 772 cu. yd. of cinders per ton.

Conclusion.—At the time the experiment station was placed in operation, no practical method was known for treating wastes produced from the manufacture of paper from straw. Where this type of waste had caused serious nuisance in the receiving bodies of water, dikes had been constructed, sometimes enclosing as many as 50 acres of land, and the ponds thus made were used as a means of storing wastes. Evaporation and seepage into the ground were taking care of the waste.

The testing station was planned and operated to modify the character of the waste so that the organic matter in the final effluent, whether in suspension or solution, would not draw heavily on the oxygen supply of the receiving body of water or kill fish, and would not form offensive sludge deposits.

It was believed that this was as far as the first studies should go, leaving the subject open for further investigation, with particular reference to the effect of the treated waste on water filtration plants purifying water from bodies of water containing the waste. This phase of the problem can be studied only after large treatment plants have been placed in operation.

The results obtained in the tests showed that strawboard waste can be treated at a reasonable cost, so that a body of water affording a dilution factor of at least 5 or 6 can receive the final effluent without danger of causing a nuisance or killing fish. In low dilutions, there will be some increase in color from the treated waste and an increase in the alkalinity, but no harmful bacteria will be added, and it is believed that domestic sewage from sprinkling filters will have more effect on a stream of water than an equal volume of the treated waste.

A complete description of the experiment station, together with a large amount of analytical data and the final recommendations in detail are given in the report of the tests.

RELATION OF WATER POWER TO THE PULP AND PAPER INDUSTRY IN CANADA

By J. B. Challies,* M. Am. Soc. C. E., and I. J. Johnston,† Esq.

The purpose of this paper is to illustrate the important assistance contributed by various branches of engineering to the development of pulp and paper manufacture in solving the unusually large power requirements of this industry. Motive power for the production of pulp and paper is vital, being almost as important as the raw material. The importance of cheap power may be judged from the fact that it requires practically 100 h. p. to make 1 ton of paper per day. In the various processes followed, the average figures of a large Canadian mill show that mechanical pulp requires 73 h. p. per ton of daily output, of which 67 h. p. is for grinding alone; sulphite pulp requires 8.7 h. p. per ton of daily output, and in other large mills as high as from 20 to 30 h. p., whereas the production of newsprint from pulp consumes 12 h. p. for the same unit output. The continuous operation of mills in this industry, usually 24 hours per day, permits an advantageous use of power, and direct water power or hydro-electric energy further allows a cheap unit cost for the amount consumed.

Importance of Pulp and Paper Industry in Canada.—The development of this industry in Canada has been rapid and the cheap power required brings it in close touch with the hydraulic resources. These facts recently led to a special survey of conditions by the Dominion Water Power Branch of the Department of the Interior, Ottawa, and the various data here presented are based largely on the information thus gathered. The remarkable expansion during the last generation may be judged from pulp and paper export trade of Canada, which is said to have amounted to only $120 in 1890 as compared with more than $100 000 000 in 1922. According to the figures of the Bureau of Statistics, for 1920, the pulp and paper industry in Canada represented a total capital investment of $347 553 333, found employment for 31 298 persons whose yearly wages and salaries amounted to $45 253 893, and the value of the products totaled $214 421 546. The fact that this industry requires large quantities of cheap power practically restricts it to hydraulic energy and the position acquired by Canada in this field rests on adequate and abundant water powers well distributed among extensive forest resources. The importance of cheap power in the manufacture of pulp and paper may further be gauged by a comparison with other Canadian industries, the power requirements of which are also large. Census figures for 1920 show that for every 100 h. p. installed in the pulp and paper industry, lumbering had only 60 h. p.; flour mills, 20 h. p.; and cotton mills, 10 h. p.

Total Power Installation.—In this paper and in Tables 5 and 6, the general term "pulp and paper mills" refers to all mills, whether they produce only pulp or paper or both these products.

Fig. 1 is a map of Canada showing the location of pulp and paper mills operated by water power or hydro-electric energy.

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† Ottawa, Ont., Canada.

The energy derived directly or indirectly from water power for the operation of pulp and paper mills in Canada amounts to 637 080 h. p., of which 476 503 h. p. is controlled directly by the pulp and paper organizations and the additional 160 577 h. p. represents purchased hydro-electric energy. This paper deals with only the important relation of water power to the pulp and paper industry and does not include data on mills in which steam is used as motive power. In Canada, the use of steam power in this industry is very limited, being prompted in most cases by special conditions, such as operations in which refuse from the manufacture of lumber can be used as fuel. The census returns of 1920 show a total steam-power installation in pulp and paper mills of only 62 400 h. p., and if the capacity of the three or four larger steam-operated mills in which special conditions obtain, is excluded, the remaining unit capacity is small.

Electric Drive in the Industry.—The electric drive is an important consideration. Of the entire water-power installation for the various mills, a total of 178 911 h. p. is converted into electrical energy before it is used to operate the pulp and paper machinery. The advantages of the electric drive are many; a uniform speed is assured, which is of primary importance in governing the quality of the product; it also allows the centralized operation of a large mill receiving power from several hydro-electric sources, and a closer study of power conditions, adjustment, and consumption is permitted. In a number of cases, the hydro-electric power is generated quite a distance from the mill in which it is used. In most instances, the mill is located at a water-power site and when gradual expansion has necessitated additional power, another near-by hydraulic site has been developed and hydro-electric energy is transmitted to the mill to permit centralized operation.

TABLE 5.—Character of Hydraulic Motive Power in Pulp and Paper Mills in Each Province.

Province.	Number of mills.	Horse-Power Installed and Purchased.					
		Turbine installation.			Purchased hydro-electric power.	Total hydro-electric Columns 4 + 6	Total from all sources.
		Direct drive.	Hydro-electric drive.	Total.			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
British Columbia...	5	27 975	20 825	48 800	.....	20 825	48 800
Ontario.....	41	89 430	81 194	170 624	72 122	153 316	242 746
Quebec.....	54	159 900	64 512	224 412	88 455	152 967	312 867
New Brunswick....	3	2 368	12 300	14 668	.....	12 300	14 668
Nova Scotia.....	10	17 919	80	17 999	.....	80	17 999
Canada.....	113	297 592	178 911	476 503	160 577	339 488	637 080

Operation from Purchased Energy.—In the Provinces of Quebec and Ontario, large power organizations supply considerable power to the pulp and paper industry. In Quebec, the Shawinigan Water and Power Company and

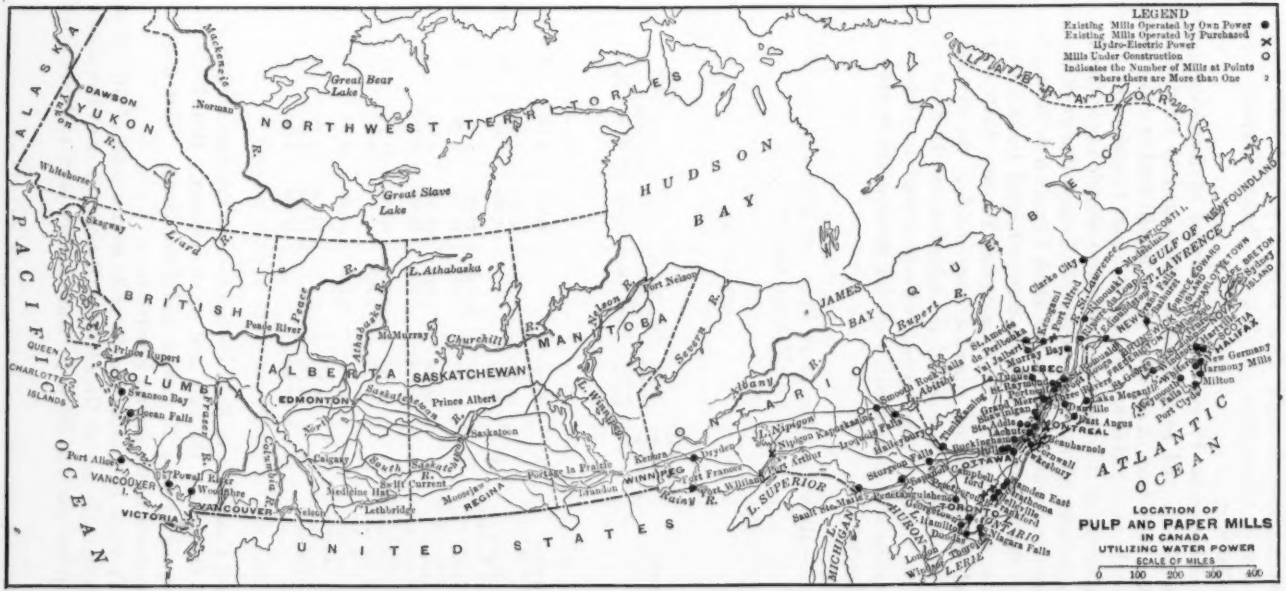

LEGEND
Existing Mills Operated by Own Power
Existing Mills Operated by Purchased Hydro-Electric Power
Mills Under Construction
Indicates the Number of Mills at Points where there are More than One
LOCATION OF
PULP AND PAPER MILLS
IN CANADA
UTILIZING WATER POWER
SCALE OF MILES
0 100 200 300 400
ALASKA
YUKON
DAWSON
NORTHWEST TERRITORIES
HUDSON BAY
BRITISH COLUMBIA
ALBERTA
SASKATCHEWAN
MANITOBA
ONTARIO
QUEBEC
NEWFOUNDLAND
LABRADOR
PACIFIC OCEAN
ATLANTIC OCEAN
UNITED STATES
GULF OF ST. LAWRENCE
VANCOUVER
VICTORIA
EDMONTON
CALGARY
REGINA
WINNIPEG
TORONTO
OTTAWA
MONTREAL
QUEBEC
HALIFAX
L. Athabaska
Great Bear Lake
Great Slave Lake
Prince Rupert
Port Arthur
Fort William
Sault Ste. Marie
L. Superior
L. Nipigon
Kenora
Dryden
Sturgeon Falls
Smooth Rock Falls
Abitibi
Iroquois Falls
Timiskaming
Thorold
Niagara Falls
Windsor
Detroit
L. Erie
L. Huron
L. Michigan
Rupert R.
Nelson R.
Churchill R.
Saskatchewan R.
Peace River
Fraser R.
Columbia R.
Mackenzie R.
Liard R.
Athabaska R.
Severn R.
Albany R.
Moose R.
Ottawa R.
St. Lawrence R.
Saguenay R.
Grand Falls
Arnprior
Carleton Place
Almonte
Perth
Smiths Falls
Brockville
Cornwall
Iroquois
Morrisburg
Merrickville
Kemptville
Prescott
Maitland
Hawkesbury
Lachute
Ste. Agathe
Ste. Rose
Ste. Thérèse
Joliette
Berthierville
Three Rivers
Shawinigan Falls
Grand Mère
La Tuque
Lake Megantic
Drummondville
Sherbrooke
Magog
Windsor Mills
Asbestos
Thetford Mines
Black Lake
Arthabaska
Victoriaville
Plessisville
Nicolet
Trois Pistoles
Rimouski
Matane
Metapedia
Campbellton
Dalhousie
Bathurst
Chatham
Newcastle
Fredericton
Woodstock
St. John
Moncton
Amherst
Truro
New Glasgow
Stellarton
Pictou
Sydney
Glace Bay
Port Hawkesbury
Antigonish
Yarmouth
Digby
Annapolis Royal
Kentville
Windsor
Lunenburg
Liverpool
Bridgewater
Milton
New Germany
Harmony Mills
Port Dufferin
Clarke City
Port Alfred
Arvida
Chicoutimi
Jonquière
Kenogami
Roberval
Dolbeau
Mistassini
Lake St. John
Alma
Riverbend
Grand Falls
Corner Brook
Botwood
Gander
Grand Falls
Windsor
Port aux Basques
Bishopsfalls
Millertown
Badger
Buchans
Bay Roberts
Harbour Grace
Carbonear
Placentia
Argentia
Holyrood
Torbay
St. John's
Conception Bay
Port de Grave
Whitehorse
Carcross
Atlin
Telegraph Creek
Stewart
Prince Rupert
Queen Charlotte Islands
Swanson Bay
Ocean Falls
Port Alice
Powell River
Woodfibre
Squamish
Lillooet
Kamloops
Revelstoke
Golden
Fernie
Cranbrook
Kimberley
Nelson
Trail
Grand Forks
Greenwood
Midway
Penticton
Kelowna
Vernon
Armstrong
Enderby
Salmon Arm
Sicamous
Revelstoke
Medicine Hat
Lethbridge
Swift Current
Moose Jaw
Prince Albert
Saskatoon
Battleford
Edmonton
Calgary
Fort Saskatchewan
Wetaskiwin
Red Deer
Drumheller
Lacombe
Ponoka
Wainwright
Vermilion
Lloydminster
Whitecourt
Peace River
Grande Prairie
Fort St. John
Dawson Creek
Hudson Hope
Fort McMurray
Fort Nelson
Fort Simpson
Fort Smith
Fort Resolution
Hay River
Fort Providence
Fort Rae
Yellowknife
Fort Norman
Norman
Mackenzie
Great Bear Lake
Great Slave Lake
L. Athabaska
Peace River
Fort Vermilion
Fort Chipewyan
Fort McMurray
Churchill R.
Nelson R.
Port Nelson
Severn R.
Albany R.
Rupert R.
JAMES BAY
Moosonee
Cochrane
Kenora
Dryden
Fort Frances
Port Arthur
Fort William
Nipigon
Longlac
Geraldton
Hearst
Kapuskasing
Smooth Rock Falls
Abitibi
Iroquois Falls
Timiskaming
Sturgeon Falls
Thorold
Niagara Falls
Windsor
Detroit
L. Erie
L. Huron
L. Michigan
Sault Ste. Marie
L. Superior
Port Arthur
Fort William
Nipigon
Kenora
Dryden
Fort Frances
Rainy R.
Winnipeg
Brandon
Portage la Prairie
Virden
Moosejaw
Regina
Swift Current
Medicine Hat
Lethbridge
Calgary
Edmonton
Saskatoon
Prince Albert
Battleford
Fort Saskatchewan
Wetaskiwin
Red Deer
Drumheller
Lacombe
Ponoka
Wainwright
Vermilion
Lloydminster
Whitecourt
Peace River
Grande Prairie
Fort St. John
Dawson Creek
Hudson Hope
Fort McMurray
Fort Nelson
Fort Simpson
Fort Smith
Fort Resolution
Hay River
Fort Providence
Fort Rae
Yellowknife
Fort Norman
Norman
Mackenzie
Great Bear Lake
Great Slave Lake
L. Athabaska
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Timiskaming
Sturgeon Falls
Thorold
Niagara Falls
Windsor
Detroit
L. Erie
L. Huron
L. Michigan
Sault Ste. Marie
L. Superior

FIG. 1.

its allied organizations supply about 80 000 h. p., and in Ontario, the Hydro-Electric Power Commission and Toronto Power Company together supply about 55 000 h. p.

Motive Power by Provinces.—The method of producing power for the operation of pulp and paper mills is summarized by Provinces in Table 5. Although a large amount of water power is still used to drive the mill equipment directly from turbines, a considerable part is utilized by first converting it to electric energy for the more convenient electric-motor drive.

Characteristics of Geographic Groups.—Table 6 illustrates the predominating features in certain districts into which Canadian mills may be arbitrarily grouped. Perhaps the greatest contrast exhibited in this table is between the class of mills found in the Niagara and Toronto, Eastern Ontario, Montreal, and Quebec groups, where a large number, the average capacity of which is small, produce much of the miscellaneous kinds of paper, whereas the British Columbia and more northerly groups in Ontario and Quebec comprise mills of large capacity, requiring much power to produce principally pulp and newsprint.

TABLE 6.—Hydraulic Power in Pulp and Paper Mills, Characteristics of Various Groups.

Groups.	Number of mills in group.	Horse-Power.				Average Daily Producing Capacity, in Tons per Mill in Group.		
		Own turbine installation.	Purchased hydro-electric.	Total for group.	Total per mill in group.	Pulp.	Paper.	
							News.	Others.
British Columbia	5	48 800	.....	48 800	9 760	147	89	6
North and Western Ontario	7	81 150	16 350	97 500	13 930	173	93	4
North Lake Huron	3	48 810	.....	48 810	16 270	268	223	....
Niagara and Toronto District	17	6 943	42 142	49 085	2 890	33	18	18
Eastern Ontario	14	33 721	13 630	47 351	3 380	42	12	17
Montreal District and Western Quebec	15	36 558	3 345	39 903	2 660	28	5	17
Eastern Townships	6	30 152	8 500	38 652	6 440	90	18	24
St. Maurice Valley and ThreeRiversDistrict.	6	21 110	66 350	87 460	14 570	268	111	27
Quebec City District	14	15 982	3 760	19 742	1 410	15	10	7
Lake St. John and Saguenay District	7	87 900	6 500	94 400	13 490	176	44	5
Lower St. Lawrence and Gulf	6	32 710	.....	32 710	5 450	72	....	....
New Brunswick	3	14 668	.....	14 668	4 890	93	....	....
Nova Scotia	10	17 999	.....	17 999	1 800	23	....	....

General Conditions in Each Province.—The Province of Quebec is slightly ahead of Ontario in the pulp and paper industry. British Columbia ranks next, followed by New Brunswick and Nova Scotia. This industry has yet to be introduced in the prairie Provinces.

The Province of Quebec has 54 mills with a total of 312 867 h. p. The three largest mills are at Grand Mère, Kenogami, and Shawinigan, each of which requires about 25 000 h. p. Among other large mills requiring between

10 000 and 25 000 h. p., are those at Hull, East Angus, Brompton, Chicoutimi, Clark City, and Cap Magdelaine.

The Province of Ontario has 41 mills with a total of 242 746 h. p., including that at Iroquois Falls, reputed to be the largest, which requires 52 000 h. p. Another mill is located at Ottawa with 28 789 h. p., and mills requiring between 10 000 and 20 000 h. p. are located at Sault Ste. Marie, Espanola, Sturgeon Falls, Thorold, Fort Frances, and Fort William.

British Columbia has 5 mills with a total of 48 800 h. p., the two largest being at Powell River and Ocean Falls with respective installations of 24 000 and 20 550 h. p.

New Brunswick has 3 mills with a total installation of 14 668 h. p., and Nova Scotia has 10 mills with 17 999 h. p.

Future Power Requirements of the Industry.—It has recently been estimated that the present demand for pulp wood requires about 20 000 acres of Canadian forests per year and, following the trend of other commodities, this consumption will probably increase rapidly. Although the reforestation now being extensively conducted, will help later to remedy this depletion, it must be noted that from 50 to 100 years are required for suitable growth. Until full results are realized, it will doubtless be necessary to extend wood pulp operations farther and farther north and the bountiful supply of Canadian water power in these regions will facilitate the fullest development of this industry.

MARINE BORERS

By W. G. Atwood,* M. Am. Soc. C. E.

Regardless of the enormous damage done by marine borers for centuries, they have been given very little systematic study. The *Transactions* of the Society contain four papers on the subject.†

The paper by Mr. Snow is a scientific study of the various borers themselves, whereas the other papers treat largely of methods of protection from their action. In various papers on harbor construction, many references to the subject are also made.

As far as is known, the studies now being carried on in the United States by the National Research Council, and in Great Britain by the Institution of Civil Engineers, are the first attempts to solve the economic problems involved in harbor construction in salt water by the close co-operation of biologists, chemists, and engineers. Both organizations are studying the borers as well as materials, methods of construction, and methods for the protection of existing structures threatened with attack by them.

Marine borers are found in fossil form and have been the source of great economic loss ever since the first structures were built in or floated on salt water. They are mentioned in the Bible where, according to a German biologist, F. Moll, the following passage occurs: "As the worm the wood so an ugly wife destroyeth her husband," and the writings of many of the early Greek and Roman scientists contain references to them. In early times, the true classification of the borers was not recognized, and those of the teredo type were thought to be worms.

The destructive borers so far as identified, belong to two families, the *Mollusca* and the *Crustacea,* shown on Fig. 2. The teredo group belongs to the family of mollusks, as do the oyster and clam, whereas the crustacean group, allied to the lobster and crab, are represented by the limnoria, sphæroma, and other similar organisms. The mollusks are also represented by the pholas, martesia, and other forms in which the body of the animal is inside the shell like that of the oyster, instead of outside of it, like the teredo.

Of the two families of borers, the mollusks are probably the more dangerous, because they enter the structure through minute holes and do their excavating out of sight, whereas the crustaceans bore comparatively shallow holes of small diameter, honeycombing the surface. The work of the latter group, therefore, can readily be discovered by inspection.

The beginning of modern literature and study of these organisms dates back to the Sixteenth Century. This study was first brought about by the destruction of the dikes of Holland, just as the beginning of the study now going on in the United States can be traced to the rapid destruction between

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† "*Teredo Navalis,* or Ship-Worm", by the late G. W. R. Bayley, M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. III (1875), p. 155; "The Preservation of Timber", by the late J. W. Putnam, Assoc. Am. Soc. C. E., Vol. IX (1880), p. 206; "Protecting Piles Against the *Teredo Navalis* on the Louisville and Nashville Railroad Company's Lines", by Richard Montfort, M. Am. Soc. C. E., Vol. XXXI (1894), p. 221; and "Marine Wood-Borers", by Charles H. Snow, M. Am. Soc. C. E., Vol. XL (1898), p. 178.

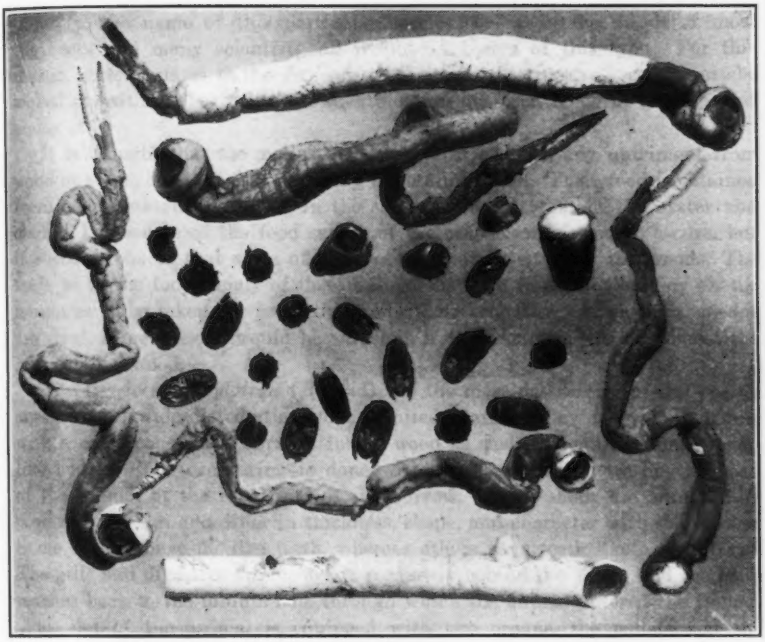


FIG. 2.—MOLLUSCAN AND CRUSTACEAN BORERS.

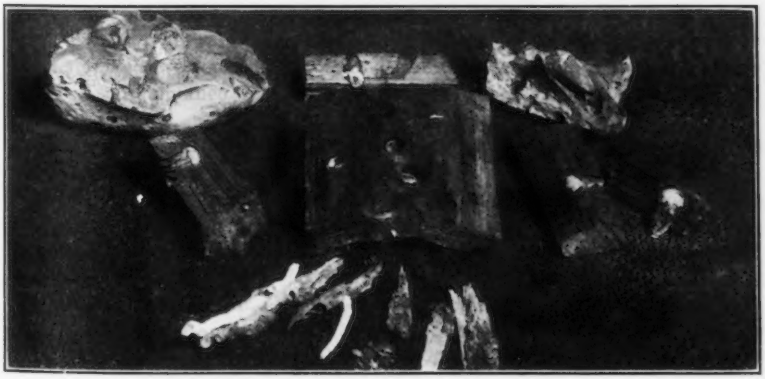


FIG. 3.—SOME ROCK AND WOOD-BORING MOLLUSKS.

1918 and 1921 of the structures in the upper part of San Francisco Bay. The destruction in Holland was caused by the species, *Teredo Navalis*, and, unfortunately, the name of this particular species covers, in the minds of most engineers and many scientists, all molluscan borers of this type. For this reason, statements as to the presence of this most destructive species must be looked on with doubt, unless the identification has been made by a competent biologist.

It is thought that the molluscan borers do not obtain any nutriment from wood or stone, but that they bore only to obtain shelter. Their food is obtained from the plankton of the sea in the same manner as that of the oyster and clam. The source of the food supply of the crustaceans is more obscure, but it seems probable that some of them obtain nutriment from the wood. The facts as to the food supply of these animals have an important bearing on the measures to be taken for protection from their activities. If chemical means are used, some poisons would be valueless if the organism did not digest the impregnated wood.

The teredo and *Xylotrya* (*Bankia*) are the most destructive of the boring organisms of wide distribution in the United States. They excavate and line, with a calcareous shell, burrows in the wood as shelters for their long worm-like bodies. The excavating is done with the shells which are on the head of the animal at the inner end of the burrow. These shells are actuated by powerful muscles and differ in thickness, shape, and character with the species. Some species have file-like teeth, whereas others have teeth like a wood rasp. The gills and digestive organs are immediately behind the shells, and the body reaches back to the minute hole through which the animal entered the timber. This end of the animal is equipped with two organs, the pallets and the syphons. The structure and shape of the pallets differ in the different species, and the characteristics of these organs form the most important means of identification. The pallets of all species have some common characteristics of form and all serve the same purpose. Each pallet is concave on the inner and convex on the outer surface, and when brought together they form a cone with the point outward. Ordinarily, the pallets project from the wood into the water and are separated. When disturbed, the animal brings the pallets together and pulls them in, thus closing the hole and shutting out enemies and harmful material.

The syphons are two muscular tubes normally extended between the pallets outside the timber. When in this position, a constant stream of water carrying food passes in through the incurrent syphon and a corresponding stream carrying the body wastes and the saw-dust resulting from the boring, passes out through the excurrent syphon. The syphons of the *Teredo Navalis* extend from 1 in. to 1½ in. outside the timber, and a badly infested pile looks as if it was covered with grey moss. When the animal is disturbed, its syphons are retracted inside of and protected by the pallets when they are closed, and no sign of the presence of the animal is shown outside the timber.

The rate of boring and of growth of the body of the animal is the same, so that the burrow is always filled. This rate varies normally from 1 in. to 2½ in. per month for the more common species. In the float of the New Jersey

Oyster Commission that was destroyed in Barnegat Bay in 1921 by the *Teredo Navalis,* there were 80 teredo per cubic inch. They were so crowded that they could not grow, but at 6 weeks of age they were about ¾ in. in length and sexually mature.

The food of these animals consists principally of the countless marine organisms known to zoologists as "plankton." Dr. C. A. Kofoid, Professor of Zoology of the University of California, states:

"The ocean in common with lakes and rivers produces each year a crop or rather a succession of crops of microscopic food known as plankton on the ocean meadows whose tonnage per acre and whose chemical composition compares with the forage crops of wild and cultivated lands."

The molluscan borers feed on this exclusively, appropriating it as it passes their doors. They never go foraging, except in the case of the free-swimming larvæ. The variation in the supply of this food is seasonable and is largely responsible for the variation in the activity of the borers. Although in infested territory there is generally sufficient food supply for the maintenance of life, there is not always enough to enable the borers to maintain maximum activity. The period of reproduction generally corresponds to that of ample food supply.

The life history of the *Teredo Navalis* and *Xylotrya* (*Bankia*) is fairly well known. The young of the *Teredo Navalis* are born as free-swimming larvæ, whereas the eggs of the *Xylotrya* are ejected by the female, fertilized, and the larvæ hatched in the water. From this time, the history of the larvæ of both species is the same. The Dutch naturalist, Sellius, states that in the body of a medium sized *Teredo Navalis,* he found 1 874 000 larvæ. The *Xylotrya* (*Bankia*) does not produce so many eggs and as its method of reproduction results in a much lower rate of survival, the *Xylotrya* (*Bankia*) is not as destructive as the *Teredo Navalis.* The larvæ of both species are free-swimming for about a month, or until the weight of the shell is sufficient to sink them. When they come in contact with a satisfactory piece of timber, they attach themselves to it by a sticky thread and commence boring.

The rapidity with which these types of borers spread, can be readily appreciated when it is realized that a single, thoroughly infested pile may contain from 50 000 to 100 000 females, each producing more than 1 000 000 larvæ per year. The depth in which these borers work is not known exactly, but it probably varies with the species. Both species under consideration will work to a depth in excess of 10 fathoms, and some species were dredged from great depths by the *Challenger* Expedition.

The martesia is another type of molluscan borer which is present, as far as known, only in the Gulf of Mexico. It greatly resembles the clam in appearance, the body of the animal being inside the shell, and is seldom more than 1 in. in length. This organism is very destructive, but as far as known, its range is limited. It differs from most of the other borers in the fact that thorough creosoting gives no protection. Very little is known of its life history.

The pholas, known on the Pacific Coast as the "rock-boring clam", somewhat resembles the martesia, except that it reaches a greater size. It bores

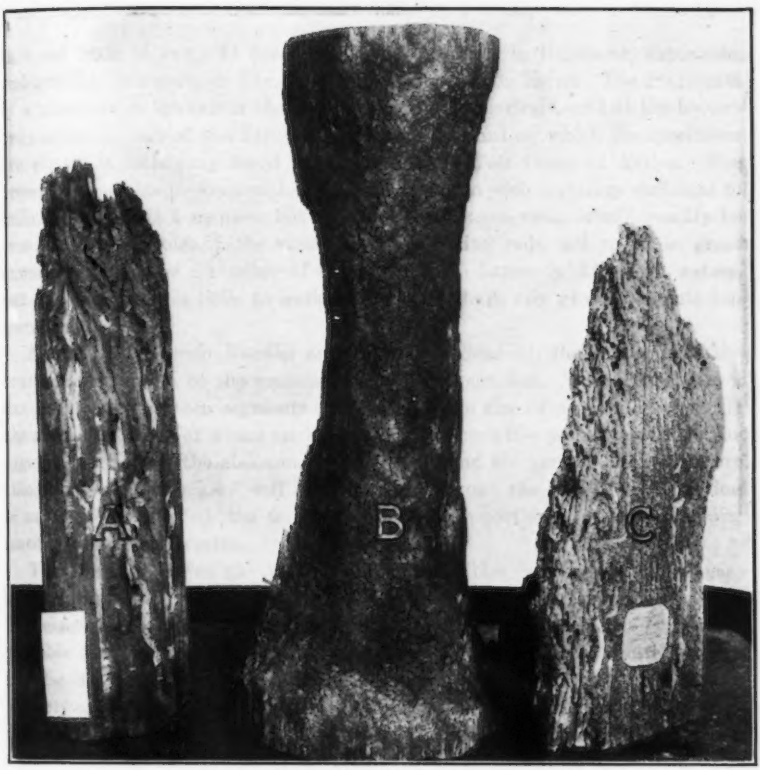


FIG. 4.—RESULTS OF LIMNORIA AND TEREDO ACTION ON PILING.

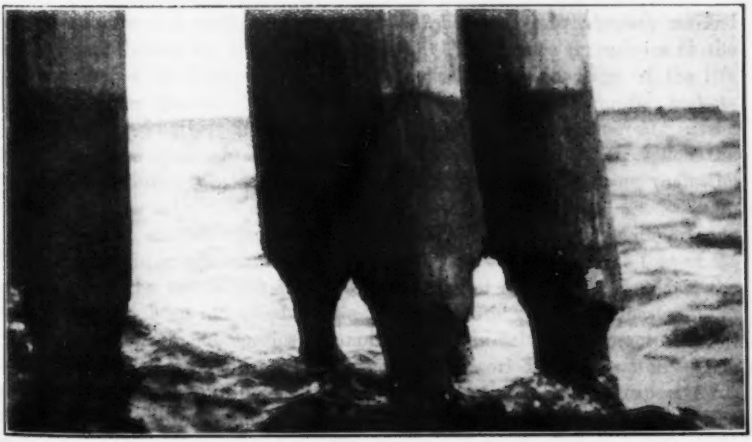
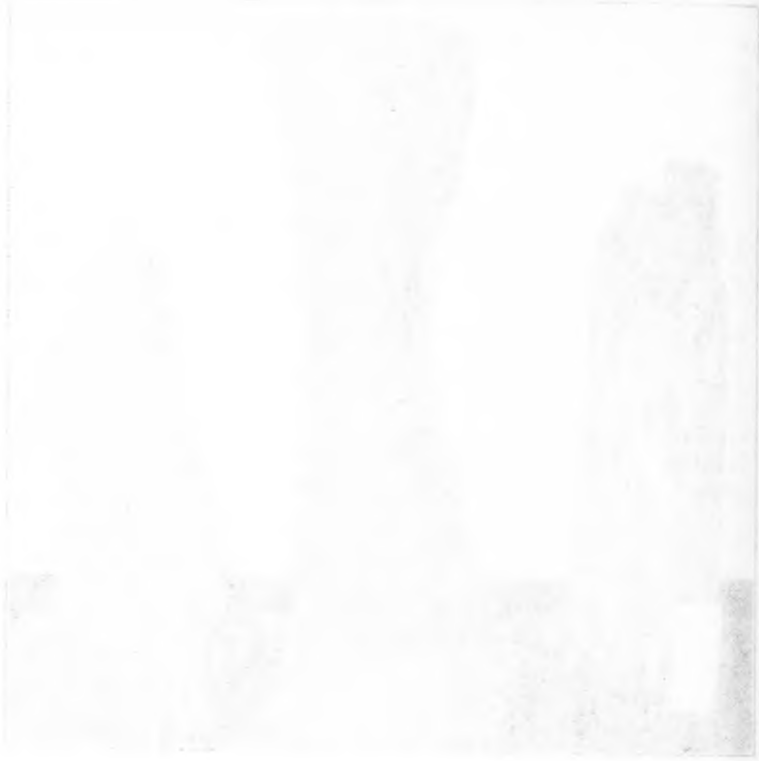


FIG. 5.—LIMNORIA ATTACK ON PILES ON ATLANTIC SEABOARD.





in wood little if any. It does bore, however, freely in limestone, sandstone, and granite, as shown on Fig. 3, at the right and left, above. The fragments of a pine pile in the center show the attack of the martesia, and at the bottom are burrow linings of the *Teredo Navalis*. The board on which the specimens are placed is mahogany bored by teredo on the West Coast of Africa. The presence of pholas in concrete has not been reported with certainty sufficient to indicate that it is a menace, but if it does attack concrete, it will readily be seen that a few holes in the vicinity of reinforcing rods will result in great damage. There are a number of other molluscan borers in American waters, but thus far there is little to indicate that they have any great economic importance.

Next to the *Teredo Navalis* and *Xylotrya* (*Bankia*), the limnoria of the crustacean group is of the greatest economic importance. This animal has a flat body with fourteen segments and is about the size of a grain of rice. It has stout legs, some of which are equipped with claws like young lobsters. The eggs are carried on the abdomen of the female and the young, when born, are able to swim and walk as well as to bore. Although the rate of reproduction is much lower than in the teredo group, the proportion of young reaching maturity is much greater.

The limnoria chews the wood with its mandibles, making small galleries about $\frac{1}{16}$ in. in diameter and $\frac{1}{2}$ in. deep. In one specimen examined in San Francisco, there were from 200 to 240 burrows per square inch. It appears probable that the limnoria uses the excavated wood as food to a limited extent. On Fig. 4, *A* is a fragment of a Southern pine pile showing *Xylotrya* attack, cut between medium and low tide, after three years of service at Biloxi, Miss. *B* shows the results of limnoria on an untreated pile after two years of service at Boca Grande, Fla. *C* shows the results of limnoria and teredo action. The last specimen was secured in March, 1922, at Manhattan Beach, N. Y.

The sphæroma is another crustacean borer similar to the limnoria in its bodily structure and method of work. It is olive or reddish brown, marked with yellow, whereas the limnoria is greyish. This species reproduces in the same manner as the limnoria; there is, however, little knowledge of the life history of either species. The sphæroma is larger than the limnoria, and its burrows are frequently $\frac{1}{2}$ in. in diameter. Its activities are generally in the tidal plane, and it is not often found at the greater depths in which the limnoria and teredo work. This species does much less damage than the limnoria, but its activities are not confined to salt water.

The chelura, another species of the crustacean borer, is of less importance than the limnoria or sphæroma. Its work has the same appearance as that of the limnoria, and the two species are frequently found together. Thus far, the chelura has only been identified on the South Atlantic and Gulf Coasts.

Marine borers of one species or another exist in salt water practically all over the world, and there are some species which work in fresh water. They are of various sizes and differ as to their living requirements and capacity for destruction, but all of them, identified so far, are classified in either the molluscan or crustacean groups. The species of teredo living in the open

sea in ships bottoms, grows to be 6 ft. in length and 1 in. in diameter, whereas the *Teredo Navalis* is seldom more than ½ in. in diameter and 8 in. long. The *Xylotrya* (*Bankia*) is sometimes ¾ in. in diameter and 3 ft. long.

The living requirements of the different species are not known with certainty. It is thought that the *Xylotrya* (*Bankia*) requires a salinity of at least 20 parts per thousand and will not work effectively where there is considerable pollution by sewage or industrial wastes, although recent reports from Seattle, Wash., seem to indicate that it may have greater resistance to pollution than has been supposed.

According to the older European reports, it appeared that the *Teredo Navalis* would not live in water with a salinity less than 9 parts per thousand, but the exhaustive experiments, under the direction of the San Francisco Bay Marine Piling Committee indicate that this species will maintain a fairly active life with a salinity as low as 4 or 5 parts per thousand, and this result has been confirmed by experiments in the biological laboratory at Plymouth, England. It has also been demonstrated that this species will live at least 28 days in fresh water, although it will not work under this condition. If salt water is supplied after this 28-day period, the animal will open its pallets, extend its syphons and begin work. In fresh running water, it does not seem to survive much longer than a week. Experiments in the San Francisco Laboratory have also shown that the *Teredo Navalis* will live and work with full efficiency in water so polluted by sewage that there is no free oxygen, and putrefaction is taking place. These experiments are corroborated by the activity of this species in Oakland Creek which is very foul, and its immunity from the effect of industrial wastes is shown by its destruction of the wharves of the Standard and Shell Oil Companies where refinery wastes were discharged into the water which, with the piles, was covered by a scum of oil. The wharves of the Shelby Smelter Company, under which there was a discharge of copper sulphate, were also destroyed.

The crustacean borers have not been thoroughly studied, but it seems probable that the limnoria requires a fairly high salinity and is not seriously affected by pollution. The sphæroma is likewise little affected by pollution, but it will live and work in either salt or fresh water. The requirements of the chelura are little known. Fig. 5 shows the typical appearance of piles attacked by limnoria.

The distribution of the more common borers, as far as is known, is as follows:

The *Xylotrya* (*Bankia fimbriata*) is found from Nova Scotia to Florida, throughout the Gulf of Mexico, and from California to Alaska.

The *Teredo Navalis* is reported by many writers on all coasts, but many reports are of doubtful accuracy. It has been identified in the vicinity of New York City and San Francisco, and in Europe, from Norway to Italy, except in the Baltic Sea east of Kiel. Some species exist in the Eastern Mediterranean and Black Sea. It is reported that ships on the blockade of Sebastopol during the Crimean War, were damaged and some of them sunk by the *Teredo Navalis*.

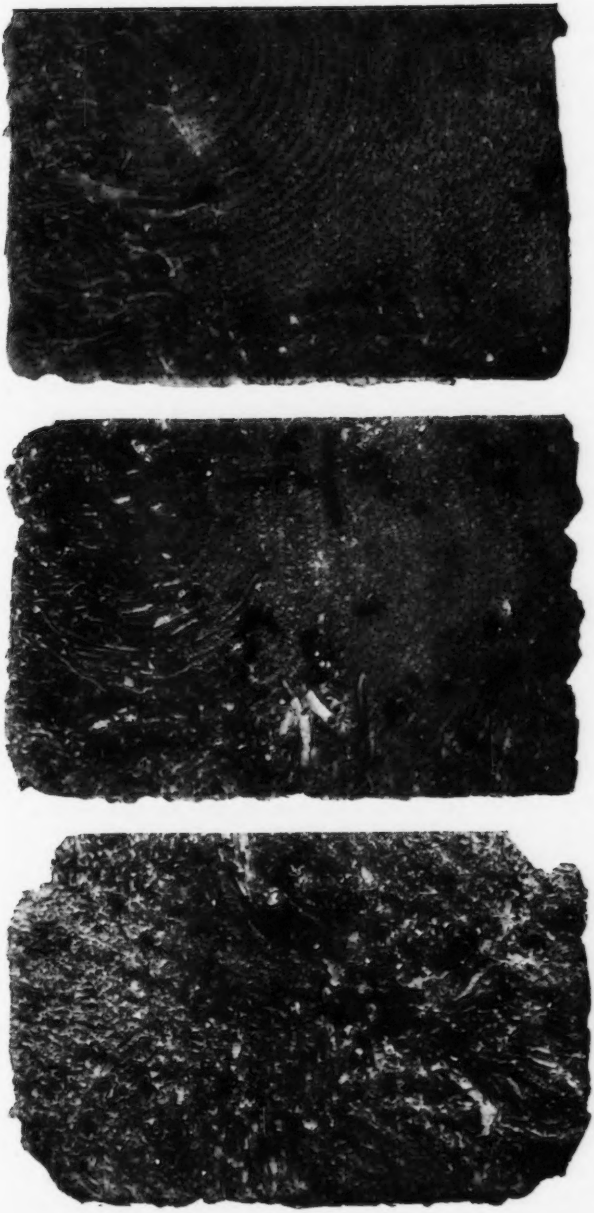


FIG. 6.—ACTION OF TEREDO ON DOUGLAS FIR TEST PIECE.

The limnoria is found on all the coasts of North America and in practically the same locations as the *Teredo Navalis* in Europe. Its maximum activity occurs near the North Cape as well as at points farther south.

The limits of the sphæroma are not absolutely defined, but it seems to be of general distribution.

The chelura has been identified so far only in the South Atlantic and Gulf of Mexico.

Some harbors seem to be immune from the activities of some or all of the borers, and the reasons for this immunity are among those facts which the National Research Council is trying to develop by its investigation. Boston Harbor, for example, has been generally immune from teredo attack and although limnoria is present, no great damage has been done. In the paper presented by Mr. Bayley in 1874, previously mentioned, it is stated that a marked reduction in the activity of teredo had taken place in the Port of New York, in the three or four preceding years. It was thought by Mr. Bayley that this was due to an increase in the sewage pollution. If this was the correct explanation, it is probable that the *Xylotrya* (*Bankia*) was the species present, as it has been demonstrated that the *Teredo Navalis* lives in water with a much higher sewage content than that of New York Harbor. The activities of the teredo may be influenced by many other factors, about which nothing is known at present.

The rate of destruction by the molluscan borers does not seem to be greatly affected by the different kinds of timber in common use for construction purposes in the United States.

Fig. 6 shows the destructive action of the *Teredo Navalis* on a 6 by 8-in. Douglas fir test piece planted on July 1, 1920, at the Southern Pacific Wharf at Port Costa, Calif. The first larvæ were discovered on August 4, and the timber was removed on December 8, 1920. Section *a* was 10 ft. 2 in., Section *b* was 6 ft. 6 in., and Section *c* was 6 in., above the mud-line. The contrast between inshore (right) and offshore (left) in the intensity of attack may be noted. The speaker had a wharf completely destroyed by *Xylotrya* in 18 months in Resurrection Bay, Alaska. W. H. Courtenay, M. Am. Soc. C. E., reports that a platform for carrying test pieces under a wharf at Pensacola, Fla., was destroyed in 30 days. Under favorable conditions, either the *Teredo Navalis* or the *Xylotrya* will destroy a 14-in. pile in 3 months.

The rate of destruction by the crustacean group is lower than that by the *Mollusca*. In waters where the limnoria is active, it seldom destroys a yellow pine or fir pile in less than a year.

The outbreak in San Francisco Bay is fully described in the valuable reports of the San Francisco Bay Marine Piling Committee issued in 1921 and 1922. As history repeats itself, one may reasonably assume that similar outbreaks may occur at other places formerly immune.

Borers, probably the *Xylotrya* (*Bankia*), had existed in the Lower Bay for many years, but were unknown in the vicinity of Mare Island and Carquinez Straits. The *Teredo Navalis* was first identified in 1914 in the long pile dikes

protecting the channel to the Mare Island Navy Yard, and the borers apparently spread slowly through these dikes in the following years. Their activities became noticeable at other points in 1919 and, by the end of 1920, they had destroyed many structures near Mare Island and in Carquinez Straits, over a distance of about 8 miles, and had been found at Antioch, 25 miles up the Sacramento River. There is a strong tidal current through Carquinez Straits, which undoubtedly aided materially in the distribution of the larvæ.

Previous trouble with borers had not been experienced at the Mare Island Navy Yard and, consequently, the Navy Department had started to build a pile highway bridge between Mare Island and Vallejo, using unprotected timber. A few days before this bridge was to be opened for traffic, it was discovered that a number of the piles were eaten off and hanging from the drift-bolts. It was necessary to re-drive the entire structure with creosoted piles. Engineers for corporations owning structures in this territory estimate that the cost of repairs of damage caused by borers to the end of 1921 was more than $15 000 000. The history of this attack clearly shows that previous immunity does not justify the assumption that such immunity is permanent. Eternal vigilance is the price of safety.

Since the beginning of history, efforts have been made to protect structures from these organisms. The Phœnicians and the Vikings charred their ships and painted them with pitch, and the galleys of Troy were sheathed with lead. The writer has recently received letters from inventors and others who think they have discovered something new. Frequently, investigation shows that these "new" ideas have been tried by the Ancients. It is necessary, therefore, that harbor engineers seeking to protect structures from the ravages of borers should thoroughly familiarize themselves with the history of the borers and the attempts to secure protection from them.

The Forest Products Laboratory of the U. S. Department of Agriculture collected a great amount of data on methods of protection and, in 1913, an exhaustive report was prepared by Mr. A. K. Armstrong, which, unfortunately, was not published. The Committee on Marine Piling Investigations of the National Research Council is continuing this study and hopes to publish the results soon. This report divides protective methods into three general classes: The first includes reports on the charring of timber, the use of built-up piles and of unbarked piles. None of these methods has any permanent value. The second includes protection by mechanical means and the third by impregnation with various chemicals.

The second method is subdivided into painting or coating with paint compounds, protection by metal sheathing, earthenware pipe, pipe manufactured of other material, concrete, or an elastic or supposedly elastic coating. On Fig. 7, *A* is a section of a Douglas fir pile taken from Seattle Harbor after 4 years of service. This pile was creosoted, but very imperfectly. *B* shows a section, 3 ft. above the mud-line, of an untreated Douglas fir pile, after 1½ years of service at Vancouver, B. C., Canada. The pile had tight bark when driven. *C* shows a section of an untreated Douglas fire pile, 6 ft. above the mud-line, after 1 year of service at Vancouver. Surface treatment with paint

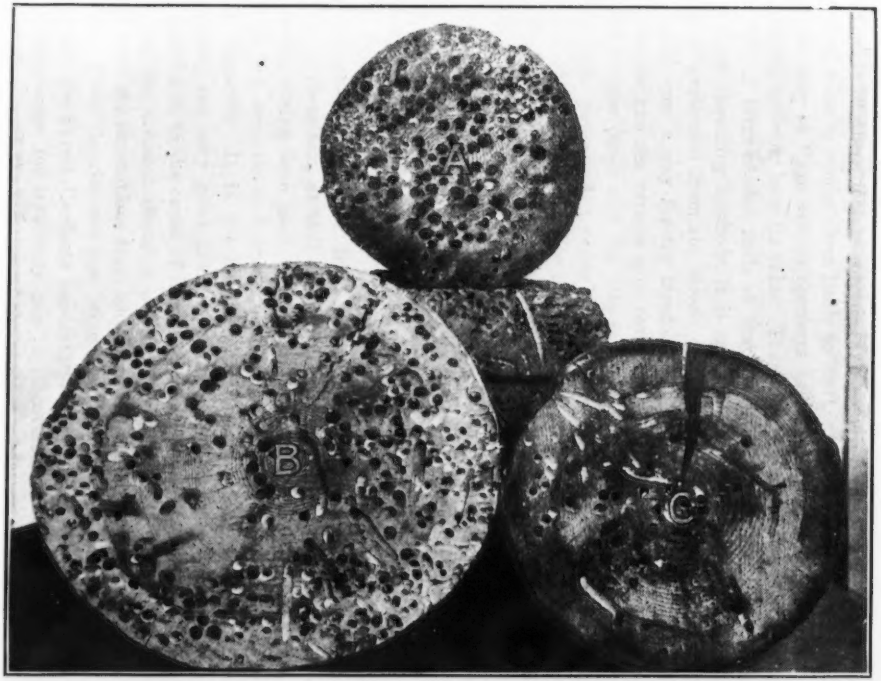


FIG. 7.—DOUGLAS FIR PILING AFTER FOUR YEARS' SERVICE.

or other material is damaged or destroyed by checking from rough handling during construction or by the abrasion or impact of floating material after it is in place. This method, although it may give temporary protection, is not permanent.

The metals used for sheathing have generally been lead, rolled iron, copper, zinc, several alloy metals, cast iron, and the process of nailing developed in Holland. All these metals corrode, and some of them, like copper, are of sufficient value to render them liable to theft. Any of these sheathings is efficient as long as there are no openings in the coating, but as soon as holes appear, the covering loses its value. The nailing process consists of driving the pile full of large-headed nails so that the surface is covered by the heads. Like other sheathing methods, it is expensive and although reported to have value as protection from the teredo, the limnoria is reported to dig out the nails and open a way for the teredo.

Concrete and the various pipe coverings are effective as long as no breaks in the surface occur and the pile is covered from the mud-line to a little above high water. A number of concrete coatings have failed, some on account of defective concrete and others because the mud-line was lowered and the borers cut off the pile between the new mud-line and the bottom of the covering.

The process of wrapping piles with a fabric impregnated with asphalt or similar material results in a pile that will resist attack until an opening is made in the coating. It is difficult to drive a pile without breaking such a coating and still more difficult to prevent damage by floating objects after it is driven.

Impregnation has been tried, many metallic salts and other supposedly poisonous chemicals, as well as creosote and mixtures of creosote and other materials, having been used. Thus far, creosote seems to give the best protection, but creosote is a trade name and does not indicate a product of fixed character. It is not known what constituent or fraction gives the protection, nor why good creosote protects timber for a time. There are two theories as to the cause of immunity from attack of some creosoted piles, one that the creosote contains a poison and the other that there is some constituent the emanations from which prevent the larvæ from landing on the timber. These theories are now being studied at two or three college laboratories and the Forest Products Laboratory.

If the reason for immunity can be discovered, a distinct step will have been made toward the solution of the problem as far as creosote is concerned. Those who believe in the inhibition theory, point to the fact that the teredo will bore freely in creosoted timber after it reaches a little size and that the martesia does not seem to be affected by it.

Some structures built with creosoted piles have had a life of 30 years or more in badly infested water, but such cases are rare. Many structures have had short life when both the creosote and the treatment were good, but the surface was damaged in construction. Careless handling of creosoted piles is probably responsible for a large number of failures.

An investigation of the resistance of various timbers was also made by the Forest Products Laboratory. It was not found that any indigenous woods would resist borer action, except, perhaps, Palmetto and some of its relatives. These timbers have little strength and decay rapidly above water level, so the fact that they resist borers is of little importance from the construction standpoint. A number of foreign timbers, principally from the Tropics, are reported to withstand attack, but great confusion exists as to the botanical identification of the species and it is difficult to obtain facts.

The Greenheart of Demarara is shown, by many reports from foreign harbors, to have long life, whereas others report its failure after a few years of service. It was used in the lock-gate sills and similar places in the Panama Canal, but was a failure and has been removed. Many reports regarding Australian turpentine wood are favorable. A small quantity of this wood was used in a wharf at Victoria, B. C., and has given excellent results for about 8 years. A number of favorable reports have also been received from the British Colonial harbors. There are also a few unfavorable reports. In regard to experience with certain kinds of Eucalyptus, it may be stated that the report mentioned does not indicate that Eucalyptus is immune from attack.

The Committee on Marine Piling Investigations of the National Research Council, with the assistance of Engineering Foundation and the co-operation of the various Government departments and the owners of water-front property, is endeavoring to determine the biological and chemical factors of the problem. It is planned, first, to discover what species exist and where; what the salinity, temperature, oxygen content, and hydrogen ion concentration are, and their effect on the different species of borers. With the assistance of Government, university, and commercial laboratories, the Committee is trying to discover why one creosote protects and another does not. The possibility of securing protection by the use of other substances is also being studied. It is hoped that, when these investigations are completed, it will be possible to draw a specification for creosote that will permit the removal of all constituents not necessary for protection and of value for other purposes, and to leave a substance that will be efficient.

The Committee is also making a study of substitutes for timber construction with the hope of issuing reports which will be of economic value to all builders and owners of water-front structures.

THE ST. MAURICE RIVER FLOW REGULATION

By O. LeFebvre,* Esq.

In 1912, the Government of the Province of Quebec adopted the policy of regulating the flow of the rivers of the Province, where storage reservoirs could be economically built and the stored water used for power purposes. The St. Maurice River was the first stream on which this policy was practiced.

The St. Maurice River rises in a series of lakes about 1 300 ft. above mean sea level, and flows from the northwest across the Laurentian Mountains into the St. Lawrence at Three Rivers. The river is about 367 miles long, has a drainage area of 17 000 sq. miles, and is broken by many falls and rapids, the principal ones of which are the following:

Name.	Height, in feet.
Les Forges	12
La Gabelle	10
Les Grès	44
Shawinigan	150
Grand 'Mère	78
La Tuque	88
Rapid Blanc	212
Rapid des Cœurs	93
Rapid Allard	45

The Shawinigan and Grand 'Mère Falls are fully developed, and that at La Tuque is partly utilized. It is expected that the falls at Les Grès and La Gabelle will be developed shortly.

Timber is the principal resource on the St. Maurice. Logs are floated down the streams to the main river, and down the latter to the different mills. The number of logs cut has been steadily on the increase until 1921.

Natural Flow of the River.—The flow of the river has been recorded at Shawinigan by the Shawinigan Water and Power Company, and daily discharge records are available from 1900 to date. The average low-water discharge is 6 000 sec-ft. The lowest discharge recorded is 5 200 sec-ft. for about a week in March, 1901, and the highest, which was recorded May 16th, 1904, was 166 000 sec-ft.

The average depth of yearly run-off from the water-shed at Shawinigan for the period 1900-21 was 21.6 in., the maximum being 26.9 in., in 1907-08, and the minimum, 15.33 in., in 1906.

Power.—The power at Shawinigan is generated under a head of 150 ft. and the low-water capacity of the plant was 82 000 continuous h. p. The machinery was designed for the discharge available during eight months of the year and the Company was disposing of the power as primary and secondary power. The total power possibilities on the river at natural low-water periods was estimated at 40 000 continuous h. p., which could be almost doubled if

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secondary power was used, but secondary power is seldom satisfactory to the customer and must be disposed of at a low price. As the demand increases, the companies on the St. Maurice look for additional power by a more regulated flow of the river.

In 1909, the power and lumbering interests organized the St. Maurice Hydraulic Company which was authorized by the Government to complete certain storage schemes on the Manouane River. Three small dams were constructed from 1909 to 1911 and the reservoirs thus created have a total capacity of 590 sq. mile-ft. The water from these reservoirs was used during summer low water and during the winter. The minimum flow at Shawinigan was thereby raised to nearly 8 000 sec-ft.

These Manouane reservoirs proved the feasibility of obtaining water from reservoirs 200 miles from the generating plant, even under most severe winter conditions. In 1912, the St. Maurice Hydraulic Company applied to the Legislature for authority to carry out storage on a much larger scale in the series of lakes which form the upper reaches of the St. Maurice River. The Quebec Government decided, however, that it would be better policy for the Province to complete the scheme, and the Quebec Streams Commission, therefore, was asked to investigate and report on the possibilities of storage in the upper lakes.

Two alternative schemes were considered: Five small dams, each controlling a series of lakes and giving partial control, or one big dam giving complete control. The latter scheme was favored and recommended by the Commission.

It was found that by building a dam near the head of La Loutre Falls, and raising the water 47 ft. above natural low water, the capacity of the reservoir would be 5 722 sq. mile-ft. The area of the water-shed was 3 650 sq. miles, and the proposed reservoir would have capacity for 19 in. of run-off from the basin.

After investigations at the site, plans were prepared and several types of dams were considered. The gravity type was recommended by Edward Wegmann, M. Am. Soc. C. E.

The design provided for 50 000 lb. per lin. ft. of ice pressure, acting at the spillway level. No ice has formed opposite the gate section within 50 ft. from the dam, and it is doubtful whether any great ice thrust has been exerted against the structure.

The dam is 240 miles from the St. Lawrence River and controls the water from a drainage area of 3 650 sq. miles. The aggregate area of the lakes forming the reservoir was 209 sq. miles under natural conditions. To-day, they form one single body of water with an area of more than 300 sq. miles.

The dam was completed in December, 1917, and has been operated to regulate the flow of the St. Maurice at 12 000 sec-ft. at Shawinigan since January 1, 1918.

Additional Power.—The benefits to be derived from regulation were calculated from the flow curve of 1906, which was the lowest year for the period, 1900 to 1912. A duration curve was made and the water necessary to make up the deficit for a minimum flow of 12 000 sec-ft. was calculated. Distributed

uniformly over the whole year, the power increase at Shawinigan totals 32 250 h. p.-years. At Grand 'Mère, where the water is being used under a head of 75 ft., the power increase is one-half that at Shawinigan.

In order to ascertain whether the quantity of water available in the Gouin Reservoir would be sufficient for a regulation of 12 000 sec-ft. at Shawinigan, investigations have been conducted since the latter part of April, 1913, when flow records were kept at the site of the Gouin Dam. The quantity of water measured at the dam site was taken as the supply from the dam and that required at Shawinigan to increase the flow to 12 000 sec-ft. was taken as the demand.

As it requires from 10 to 12 days for the water from the reservoir to reach Shawinigan the calculations were based on a loss of 25% from evaporation and on other losses. Similar calculations were made for a regulation to 14 000 sec-ft. The curves show that such regulation is possible, but if a loss of 25% should occur, the reservoir would not be large enough. Since the dam has been in operation, it has been noted that the loss is negligible, being only about 5% in the winter.

Discharge Control.—The discharge of the river is controlled at three points, namely, at Weymontachingue, 50 miles below the dam, at La Tuque, 155 miles from the dam, and at Shawinigan. The water level at Weymontachingue and at La Tuque is recorded daily and the information wired to the office of the Quebec Streams Commission. Similar information is obtained from the Shawinigan Water and Power Company regarding the conditions at Shawinigan and Grand 'Mère. The points of control are also provided with rain gauges. In this way, a good idea may be gained as to the period it will be necessary to draw on the storage.

Ground Storage.—By raising the water 40 ft. above natural high water at La Loutre, the different lakes form one body of water, more than 300 sq. miles in area. As a great deal of the area covered is pervious, much ground storage must take place. The natural run-off, as a rule, is a minimum in March. Before the water was raised, the average for the winter months of three years was as follows:

January	291 sq. mile-ft.
February	219 " " "
March	215 " " "

Since the reservoir has been operated, the run-off in March and February is much larger than in previous years, as shown by the comparison given in Table 7. This cannot be explained otherwise than by ground storage.

TABLE 7.

Months.	1919.	1920.	Average of 3 years before operations began.
March.....	450 sq. mile-ft.	785 sq. mile-ft.	215 sq. mile-ft.
February	284 "	751 "	219 "

Land Clearing.—The area flooded by the reservoir has been estimated at 100 sq. miles, most of which is forest. This flooded area was not cleared on account of the cost, but it is not expected that much of the floating dead wood will reach the dam.

Manouane Dams.—Mention has been made of the three dams on the Manouane River. These dams have been acquired by the Commission and are being operated together with the Gouin Dam for the regulation of the St. Maurice. The dams are of the ordinary wooden crib type, filled with stone, their sluice openings being controlled by stop-logs. The combined capacity of the reservoirs formed is nearly 590 sq. mile-ft. The combined drainage area is 1 250 sq. miles. The supply of water from the basin is, therefore, much larger than the capacity of the three reservoirs, which overflow in the spring and are generally emptied during the dry summer months.

Cost.—The cost of the whole work was $2 500 000, or $400 per sq. mile-ft. of water. The yearly revenue is at present $215 000, which will be increased when new water power is developed.

This regulation work has been a success and, the amount of power now generated on the St. Maurice River is almost 500 000 h.p. Nearly 500 000 additional h.p. could be developed. It is believed that these works will make the St. Maurice River one of the most important sources of water-power on this Continent.

SHORE PROTECTION AND HARBOR DEVELOPMENT WORK ON THE NEW ENGLAND COAST.

By Frank W. Hodgdon,* M. Am. Soc. C. E.

The New England coast consists of ledges, sand beaches, and bluffs composed of glacial drift, and the beaches and bluffs are gradually washing away.

The sea acts in two ways: First, sand and gravel, stirred up by the waves, are carried over the top of the beach, thus filling the low land back of it and forcing the shore line inland.

Where the land back of the beach is high, the waves drift the material along the shore until a low beach with low land back of it is reached. Here, the material is either washed over the beach on to the low land, or gradually drifts into the sea and forms shoals, as at the southeasterly part of Cape Cod Bay where they extend more than a mile from the shore.

Many different forms of protective work have been used in the endeavor to prevent further erosion. In some places, timber bulkheads were built at or above the high-water mark. A number have been back-filled, but in almost every case the timber structures have been either broken down or undermined. The large islands which form much of the protection to Boston Harbor, were early protected with heavy granite sea-walls and rip-rap by the Federal Government.

In 1828, a wall was built to protect the headlands of Deer Island and, in 1843, one was built at Lovells Island. Walls were built later at the Brewster Islands. In 1871, walls were built at Point Allerton, Long, and Gallops Islands. These later walls on the outer shore are about at the high-water line and their foundations are about at mean sea level. The tidal range is about 9.5 ft. and the top of the wall is about 22.5 ft. above mean low water. The wall is 5 ft. thick at the top and 9 ft. at the base, with a footing 12.5 ft. wide. The face is of granite and the backing and footing are of concrete. Adjoining the wall at the level of its top a pavement, 15 ft. wide, of granite slabs has been placed on the fill to prevent wash from waves which break over the top. The beach in front of the wall is also protected with a rip-rap of granite grout. These walls, although requiring repairs from time to time, have effectively protected the cliffs from erosion.

At the bluffs in Scituate and Chatham, the Commonwealth has deposited heavy granite grout from the high-water mark to 12 or 15 ft. above it. Sections of this protection at the Scituate Third Cliff have been in place for 14 years. At Chatham, at the bluffs in front of the lighthouses at East and West Chop, on Martha's Vineyard, ridges of stone have been placed about at high-water mark for a considerable distance on each side of the lighthouse. This has been an effective protection.

At Cotuit and Osterville, on the southerly shore of Massachusetts, facing Nantucket Sound, the bluff which is about 10 ft. high, is being eroded rapidly by the action of the sea. The owners have constructed timber bulkheads along the shore just above high water and, at intervals of 30 to 50 ft.,

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short timber spurs or jetties have been extended, which have prevented the sand from drifting along the beach, and a barrier has been formed, which has stopped the sea from making further inroads. These structures require constant attention as they are usually built of 2-in. plank spiked to 4-in. square stakes, the posts of the bulkhead being of cedar, 6 or 8 in. in diameter. This work, although costing considerable for upkeep, has been effective. The water in front of these bulkheads is shallow for ¼ or ½ mile. The range of the tide is generally less than 5 ft., and the waves are not heavy.

To protect the beaches at various points along the shore, walls with spur jetties from 50 to 100 ft. apart, have been built of concrete. The walls are located on the crest of the beach or between the crest and the high-water mark and the jetties project 1 to 2 ft. above the level of the beach. These walls extend about 22 ft. above low water or 12 ft. above high water and are built with a lip projecting about 6 in. at the top to deflect the sea. On the Scituate shore, some of the first walls had no lip and extended only 18 to 20 ft. above mean low water. At this elevation, the sea broke over the top and in places, large quantities of sand and shingle were washed over the wall. The beach was thus reduced, enabling the sea to undermine the wall in places so that it collapsed. It has been rebuilt to the elevation of 22 ft. with a projecting lip.

The harbor improvements may be divided into two parts: The improvement and protection of the channels, and the construction of wharves and docks. In the larger harbors, the U. S. Government has enlarged and deepened the channels and protected the entrances.

Boston Harbor.—In Boston Harbor, the principal work has been the dredging of the channels and the removal of isolated rocks and extensive ledges. The material, which is mostly clay, gravel, hardpan, and rock, has been excavated largely with scoop dredges. At first, the excavated material was deposited in the harbor, but recently most of it has been dumped in deep water outside the harbor. Inside the harbor, a large quantity of material was used for filling at South and East Boston where the Commonwealth has undertaken large reclamation plans. There are three entrance channels in Boston Harbor. The original or southerly channel just south of Boston Light crosses the entrance of the harbor about at right angles to the direction of the current caused by the tide. It originally had a depth of about 18 ft. at low water and was partly obstructed by rock or large boulders which have been removed. This channel has been dredged to a depth of 27 ft. at mean low water, with a width of nearly 1 000 ft. The Broad Sound Channel, which was next opened, has a depth of 30 ft. and a width of 1 200 ft.

The third or northwesterly channel extends straight from the sea into President Roads and has a depth of 35 ft. at mean low water and a width of 1 500 ft. President Roads is a large area with a depth of water exceeding 40 ft. at low tide, into which these three channels lead. Extending from President Roads to the wharves at the city is a channel 35 ft. deep and 1 200 ft. wide, which originally was not more than 18 ft. deep at low tide. The excavation was mainly of silt and clay with some hardpan and considerable ledge, most of

the rock being slate. The character of the bottom of the harbor is such that there is very little shoaling caused by shifting material. The currents generally are less than 1½ miles per hour.

Leading to various points in the harbor are a number of subsidiary channels which have been dredged to depths ranging from 6 to 30 ft. at mean low water. In the inner harbor, a large number of wharves were constructed at an early date and until about 1870 were practically all owned by private individuals. Since that time, the Commonwealth has improved the flats at South Boston and, later, those at East Boston.

About 600 acres of flats extend from the shore of South Boston from ½ to 1 mile and about 2 miles in length from the city to Castle Island. This area has been filled by the Commonwealth and its grantors, mainly with material excavated from the harbor channels, and on it are the wharf terminals of the New York, New Haven and Hartford Railroad, and the Commonwealth Dry Dock, 1 200 ft. long, 35 ft. over the sill at low tide and 120 ft. wide in the entrance, which was built by the Commonwealth and sold to the United States Navy Department in 1919. Alongside this dock is the area sold to the United States Army on which the Army Base was erected at a cost of about $25 000 000. The Commonwealth Pier, which is used by the largest merchant ships, and the Fish Pier, on which is the greatest fish industry in the United States, are also located here.

The Commonwealth Pier is 1 200 ft. long and 400 ft. wide with a depth of 40 ft. in the slips at low tide, the mean range of tide being nearly 10 ft. The pier itself is constructed of a granite sea-wall, 16 ft. high, founded on piles which are cut off at mean low water. The area enclosed by this wall, which is 2 700 ft. long, is filled with material excavated from the docks. Outside the sea-wall is a pile platform, 50 ft. wide, covering the slope of the excavation from the bottom of the dock to the foundation of the sea-wall at low water. This slope, which is 1 : 1.5, is covered with rip-rap. On this, as a foundation, has been built a steel and concrete two-story shed with a heavy masonry front or head-house. On this pier, in addition to the spaces devoted to cargo, are accommodations for handling from 1 000 to 2 000 passengers at a time, with facilities for all the necessary examinations of the passengers and baggage. The passenger facilities are on the second floor, from which a viaduct leads to Summer Street.

The Fish Pier is a solid-fill wharf. The filling is retained by a granite wall, 41 ft. high, founded on rip-rap deposited in a trench in the bottom of the harbor. The water in the slips alongside is 23 ft. deep at mean low water. The pier is 1 200 ft. long and 300 ft. wide. The vessels lie up against the granite wall, on the face of which are oak fenders. The surface of the wharf outside the buildings is paved with granite blocks on a concrete base and the buildings are of concrete. In addition to the stores for handling fresh fish, a large cold storage plant has been built in which the surplus fish are stored awaiting a market.

The wharves at the New Haven Terminal are similar to the Commonwealth Pier, except that the floors of the sheds are of timber instead of

concrete and the sheds are of wood instead of steel and concrete. All piles of the platforms in both terminals are of oak.

At the Army Base, the piles of the wharf, which support a heavy concrete floor, are of pine. The buildings are of concrete and steel. Those on the solid portions are supported on concrete cylinders sunk into the clay, and the two-story sheds on the wharf are on the pine pile foundations. The sheds on the wharf are connected with the building on the solid portion by bridges over the driveway and tracks between the buildings. This wharf is equipped with four cranes of the semi-portal bridge type, each of which has a capacity of 4 tons at a distance of 29 ft. from the face of the wharf.

The early wharves in Boston Harbor were built of stone-filled timber cribs enclosing areas which were filled with earth. Granite walls were used later in place of cribs and, in order to secure a greater depth of water alongside, the solid wharves were extended by building pile piers. In some cases, the granite walls were built on pile foundations. Some of the pile piers or platforms were built with timber floors, but, in others, the piles were capped near high-water level. Round poles were laid on the caps and covered with marsh sods. On this, gravel was filled to the level of the wharf which was 14 to 16 ft. above mean low water. The filling was held in at the face of the wharf by timber bulkheads tied back to the capping of the piles. Thus, the timber work and piles, except the bulkheads which retained the filling, were kept wet and did not decay, and the cost of maintenance was small. A few wharves were built with granite walls extending about 20 ft. below low water, but most of the walls were founded on hard bottom or on piles at about low-water level,

Other Harbors.—Massachusetts has a number of small harbors which have been improved by dredging the channels and anchorage basins and by building breakwaters and jetties to protect their entrances. The principal work which was undertaken by the Federal Government was the construction of a harbor of refuge at Cape Ann, where a heavy granite breakwater was built to enclose a large deep-water bay, known as Sandy Bay, at the outer end of the Cape. Up to low water, the breakwater consists of a ridge of large blocks of granite quarry grout. The superstructure consists of a facing of large split dimension stone backed up with quarry grout. About 6 000 ft. of the substructure has been built and about 900 ft. of the superstructure.

The slope of the faces of the substructure is 1 : 2 and of the superstructure 1 : 1. The facing of the superstructure consists of blocks weighing more than 20 tons each. The top of the finished breakwater is 22 ft. above mean low water and is 20 ft. wide. As the stones are somewhat displaced in heavy weather, the maintenance of the superstructure may be costly. Owing to changes in means of navigation, there is now much less need of a breakwater in this locality, and it is probable that this one will not be completed soon.

On the sandy shores like those at the mouth of the Merrimac River, stone jetties have been built to confine the current and direct it so that a channel will be scoured through the bar. To confine the current and prevent the waves from washing sand through the jetty into the channel, these jetties have been built of a facing of granite quarry grout with a core of chip

stone. They extend out to deep water, in order that the sand scoured out will be deposited there.

The Commonwealth has built a number of breakwaters to protect small harbors, such as Apponaganset near New Bedford and at Vineyard Haven on Martha's Vineyard. These breakwaters are ridges of granite quarry grout 5 ft. wide on top, with side slopes of 1 : 1.5, the top being from 4 to 5 ft. above high water. These breakwaters protect anchorage areas used by small boats and yachts.

Because of insufficient funds, most of the work done by the Commonwealth cannot be completed at one time and, in a number of cases, it has been practically abandoned after being partly completed. This has given an opportunity to study the effect of the work in various stages.

In some cases, channels constructed through sandy beaches, as at the easterly end of the Cape Cod Canal, have a long jetty on the side of the opening from which the sand drifts, and a short jetty on the opposite side. The sand is thus prevented from entering the channel, and the beach back of the jetty is built up.

In some instances, the funds were small, and a jetty was built on the leeward side of the opening, in which case, the current was relied on to scour a channel. This has resulted in a narrow channel through the beach, which to the leeward has built out and up, and shoals have formed in front of the channel. The practice of placing the jetty on the windward side of the opening and dredging the channel has been more satisfactory.

Cape Cod Canal.—The largest private undertaking in Massachusetts was the construction of the Cape Cod Canal. This project had been under consideration almost since the settlement of the country. Surveys were made from time to time up to 1860, when an exhaustive report was presented to the State Legislature. From 1870 to 1889, many charters were granted by Massachusetts to build canals from Buzzards Bay to Barnstable Bay and, about 1880, two attempts were made to excavate a canal, both of which failed for lack of financial support. In 1889, the Boston, Cape Cod, and New York Canal Company was incorporated, which later secured the co-operation of Mr. August Belmont of New York City. On August 19th, 1907, the construction of a canal was begun, which was opened to traffic on July 29th, 1914.

The canal is 8 miles long, has a depth of 25 ft. at mean low water, a bottom width of 100 ft., and side slopes of 1:2 in the main part of the canal. The bottom width of the easterly approach channel is 300 ft. and that of the westerly, or Buzzards Bay, approach is 150 ft. The sides of the canal are rip-rapped from 6 ft. below mean low water to 6 ft. above mean high water to protect the banks from the wash of passing vessels.

The canal is spanned by three draw-bridges having openings 140 ft. wide. The two highway bridges have a clearance of 30 ft. above extreme high water, that of the railroad bridge being 2.5 ft. The canal is electrically lighted throughout, and the draws in the bridges are operated by electricity.

The tidal currents in the canal are quite strong, the mean tidal range at the easterly end being 9.4 ft. and at the westerly end, 4.2 ft. High water

at the westerly end is about three hours later than at the easterly end so that the direction of flow changes quickly. The velocity of the current is from 3 to 5 miles per hour in the narrow section.

The easterly entrance is protected by a stone jetty about 3 000 ft. long on the northerly side. The westerly end is so well protected at the head of Buzzards Bay that no jetties were required, but a channel about 6 miles long had to be dredged in the Bay to reach deep water, thus making the distance from deep water in Barnstable Bay to deep water in Buzzards Bay about 14 miles.

At each end of the canal, where the section widens, shoals occur, which are removed by dredging. That part of the canal above tide level was largely excavated by steam shovels and drag-line scrapers. The lower part was excavated mostly by dipper dredges as the material contained many boulders. This material was dumped in deep water outside the canal. A small part was excavated by hydraulic dredges. This material was dumped on the marshes and other low lands on the banks of the canal.

Because of the swift current, the advisability of placing a lock in the canal was discussed. It was decided, however, not to install it, and the canal is being operated successfully.

DIFFICULT FOUNDATION PROBLEMS FOR PISCATAQUA BRIDGE AT PORTSMOUTH, N. H.

By J. W. Rollins, Jr.,* M. Am. Soc. C. E.

The Piscataqua River is not very long, but has great possibilities for trouble. The tidal flow through the 900-ft. channel at Portsmouth is very great, the ebb tide having at times a velocity of 8 miles per hour.

In 1917, the New Hampshire Legislature authorized the appointment of a commission to investigate the feasibility of the construction of a highway bridge across the river.

The Commission made a favorable report to the Legislature of 1919, and Governor John H. Bartlett of New Hampshire, advocated the construction of a bridge in accordance with the report. The Legislature appropriated $500 000 for its share of the cost of the bridge. The Maine Legislature of 1919 appropriated $500 000 for the work, and the Federal Government was also induced to appropriate $500 000.

Under the legislation enacted, a new commission was organized consisting of Secretary of the Navy Daniels, Governor John H. Bartlett, of New Hampshire, and Governor Carl E. Milliken, of Maine. Each of the Commissioners appointed an engineer to represent him, Secretary Daniels appointing Commander E. H. Brownell, U. S. N., M. Am. Soc. C. E.; Governor Bartlett, W. A. Grover, M. Am. Soc. C. E. of Dover, N. H.; and Governor Milliken, Mr. Lewis B. Jones, Bridge Engineer of the Maine Highway Commission. Mr. Jones was killed later in an elevator accident in Portsmouth, and Walter H. Norris, Assoc. M. Am. Soc. C. E., Bridge Engineer of the Maine Central Railroad, was appointed to take his place.

The Board of Engineers decided that the Portsmouth end of the bridge should be at Daniel and State Streets, and that it should cross the river at approximately right angles to the Kittery side, on Badgers Island. In September and October, 1919, investigations were made of the bottom of the river at the selected site, and this work was continued in May, June, and July, 1920.

Owing to the contour of the channel, the flood tide is on the Kittery side of the river, and the ebb tide on the Portsmouth side. These tides, acting in different channels, form great eddies the force of which cannot be estimated. The normal rise and fall of the tide is 9 ft., and this may be greatly affected by heavy winds.

The river bottom is covered with a layer of cobble-stones and gravel under which is a stratum of soft clay or mud, 3 to 10 ft. thick; next, there is a layer of hardpan from 6 to 10 ft. thick, overlying rock. The hardpan is so dense that a 5-ton orange-peel bucket would not touch it, and in the excavation in caissons it had to be loosened with pneumatic hammers.

The borings for the piers and abutments were made with drills, by wash-borings, and with well drills, the latter being used for the final determination

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of the piers. The borings and soundings for the Portsmouth Abutment and the North Pier were close to actual conditions as determined by excavation, the North Pier being on rock at Elevation — 80.

On September 14, 1920, proposals were asked for the construction of the foundations and superstructure, in two separate bids. The two bids exceeded the amount available, $1 500 000, and the contracts were not awarded.

The original plans called for piers about 100 ft. long, with large sloping ice-breakers on the up-stream end, with a double Strauss lift bridge of 300 ft. span. As the authorities were anxious to complete the bridge, further studies were made to reduce the cost. A Waddell lift span was substituted for the Strauss lift, and the piers were re-designed and made symmetrical by omitting the ice-breaker. The result of these studies was that contracts were let, the substructure being awarded to the Holbrook, Cabot and Rollins Corporation, and the superstructure to the American Bridge Company. The work was awarded in November, 1920, and the speaker's firm began immediately to get plant and material on the site.

The great problem on this job was the moorings and anchors. No information as to the force of tides or water could be found, so the opinions of shipbuilders were obtained. The result of these inquiries was that a force of 80 tons might result from the worst tidal condition. No anchors would hold against such a force; therefore, for the main up-stream and down-stream anchors, cribs about 20 ft. square and 15 ft. high were built and filled with gravel.

The cables used were of galvanized plow steel, 2½ in. in diameter, having a breaking strength of 200 tons. For the side anchors, two 40-ft. concrete piles, weighing 10 tons each, were used, with 1½-in. steel cables. Each anchor had two cables to the caisson, one fastened to the concrete air caisson, the other at the top of the first set of caisson sides, making a total of twelve cables.

The general design of the caisson was a concrete air chamber, 27 by 75 ft., with a 7-ft. clearance. The walls were 5 ft. above the deck of the chamber, and, above that, regular wooden caisson side construction was used. The caissons were built on shore, where the launching space was very narrow and the water only 9 ft. deep at low tide. The launchways were placed on a grade of 2 in. to 1 ft., and it was planned to launch the caisson sidewise.

As no one seemed to know how the caisson would behave if it was launched with the 7-ft. air chamber open, it was decided to plank it solid across the bottom. No special provision for water-tightness was made, except to use matched planking.

The launchway had four double lines of 12 by 12-in. hard pine timbers as fixed ways, with a double set of 12 by 12-in. hard pine timbers as sliding ways. The fixed ways were spaced 4 in. apart, and the movable ways had a 4-in. guide bolted between them, thus forming a fixed guide to keep the caisson straight. The fixed and movable ways were bolted together with 4 by 12-in. splices on the inshore end. The caisson was launched by cutting these 4 by 12-in. strips.

The concrete air chambers were designed as the foundations of the piers, and on account of the great depth of the water, the relation between the mass of masonry in the pier and the cubic area of the caisson, was very close. For

this reason, it was not considered safe to build the piers solid above the roof of the air chamber designed to carry the hydrostatic pressure to the full depth of water at the pier site. Consequently, the piers were built with side-walls 5 ft. thick at the bottom and 3 ft. thick at Elevation — 80, with three lateral cross-walls and one longitudinal wall in the center. Excessive pressure on the caisson sides was avoided with this distribution of concrete. This form of construction later proved to be a great aid in sinking the caissons. After some experiments, the caissons were built to their full height, 66 ft. above the concrete. They were loaded to draw 40 ft. of water before being towed to the bridge site.

It was decided to build the North Pier first, owing to its more favorable position as to depth of water and tidal current. At this pier, the ledge was at Elevation ± 25 at the high corner and Elevation + 18 at the low corner, and was overlaid by 8 to 12 ft. of clay, with some hardpan. This overburden was dredged to the hardpan, about 3 ft. above the ledge. The caisson for this pier was built with one set of caisson sides, 22 ft. above the concrete air chamber. It was loaded to draw about 25 ft. of water and towed to the site.

The northerly side anchorages were on the ledge of Badgers Island, whereas all the others were crib or concrete anchors in the river. The flood tide struck the caisson about at right angles to its down-stream northerly side. No great strain was anticipated for the side anchors, as the ebb tide apparently was clear of the caisson. However, it was soon noted that the direction and force of the ebb tide had to be considered. The caisson would rest peacefully for a long time, and then some great swirl or eddy would strike it, and send it inshore, dragging the 40-ton anchors on the river side. With the outside line slacked, the flood tide would send the caisson back, bringing up hard on the inshore cables fastened to the ledge.

With 2 000 tons moving and "bringing up" hard on the inshore cables, it was evident that something must give way, as the side cables were not fastened to the caisson in a manner to withstand these shocks. The caisson, therefore, was taken back to the wharf and another set of sides 22 ft. high were built. A second set of fastenings was also built in it, and a second trip to the site was made.

The second attempt to land the caisson brought the same troubles as the first. The eddies on the ebb tide carried away the outboard anchors, moving the caisson 50 ft. inshore and landing it on the bottom of the river. It was finally decided that the bottom of the river was the only safe place for it.

With a hollow caisson, it was possible to flood the pockets with water and keep it on the bottom, and then, at low tide, pump out the water in one pocket and place the concrete. When a greater depth of water was needed to float the caisson, it was moved at high tide out into the deeper water. When the walls were built to Elevation 80, and concrete sufficient to hold the caisson in its true position on the bottom had been placed, it was moved out at high tide, and then sunk by filling the pockets with water. On the following slack tide, all the lines were "hitched up" and, at the next high tide, the caisson was floated by pumping out the water, lined up, and sunk again in its true position, which was within 3 in. of the line. Afterward, the caisson was kept on the bottom and sunk to rock.

The Portsmouth Abutment was built at the same time as the North Pier. A pier design was substituted for the Kittery Abutment. It looked simple on the plan—a sloping foundation on rock at an angle of about 30°, the front edge being shown at Elevation 74. Steel sheeting was driven below Elevation 74, 5 ft. in front of the line of the foundation, and the excavation was begun. Trouble soon developed, as the rock "dropped off" almost perpendicularly in the middle of the coffer-dam.

The sheeting was undermined and then driven down to low water. The excavation was made with a bucket, a sand-pump, and divers. The ledge was finally cleaned over the greater part of the area. Concrete was deposited under water, the entire excavation being filled to the sheeting. The excavation having been sealed, the coffer-dam was unwatered after a supplementary wooden dam had been built from the steel sheeting. The masonry was finished after a delay of several weeks.

At the South Pier, the bottom of the river sloped from about 1 ft. to 8 ft., and the rock, according to the soundings and borings, was at Elevations 29 and 26 on the inshore side, and Elevations 23.7 and 19.8 on the outer side. It was intended, therefore, to land the caisson on rock at about Elevation 25. Meanwhile, all the soft material possible had been dredged and the site of the pier had been leveled at about Elevation 30; this meant that the inner side would be hardpan and the outer side, the original bottom, that is, soft material.

It was feared that the caisson, when landed, would either go out of level toward the river, or would slide out of position into the river. To avoid this, a windrow of crushed stone, about 3 ft. wide and 2 ft. higher than the hardpan bottom on the inside, was deposited along the outside edge. The caisson was towed to its site and sunk very near to position. On the next high tide, it was floated and pulled into a position 1 ft. inside of its true line. Within 24 hours after it had been landed, it had slid about 12 in. out into the river. It was again floated and placed 24 in. inshore and landed. It again moved about 12 in. and stopped. Rip-rap weighing 1 000 tons was then piled on the outside and no further movement took place. The water was pumped out and the concrete put in. The caisson was permanently landed about 8 in. out of position inshore.

The crushed stone kept the caisson about 1 ft. out of level inshore, until it was landed on hard bottom. At Elevation 26, the supposed rock level, no rock appeared. The sinking was continued to Elevation 20, and still no rock. The sides of the caisson were not high enough, and the shafting, locks, air, and other pipes, had to be built up.

The contract did not require excavation to rock, and it was contended that at Elevation 20, the foundation on hardpan was suitable. The caisson was then 19 ft. below the original bottom on the up-stream end, and 12 ft. on the down-stream end.

The excavation was continued, and the cutting edge sunk to Elevation 16, 10 ft. below the designed level. The rock was cleared off in places to Elevation 9. As this additional work was done under an air pressure of 40 lb., the cost was excessive, being about $3 000 per ft. However, the entire contract was finished in March, 1922, seven months ahead of requirements.

TESTS OF CONCRETE IN SEA WATER

By L. C. Wason,* M. Am. Soc. C. E.

In the summer of 1901, the writer was frequently at the Charlestown Navy Yard, and observed the building of Pier No. 1. The concrete, of which this pier was constructed, was mixed so dry that, when it was tamped in place, no water showed on the surface. In a few years, this pier showed serious defects, and the writer and his associates discussed with various engineers the cause of this early failure. Many reasons were advanced, but no conclusion was reached. This discussion was broadened through the technical press without obtaining a satisfactory answer, and it was finally decided to investigate the cause. After extended publicity and correspondence, a series of twenty-four test specimens, 16 in. sq. by 16 ft. long, were made of various qualities of cement. Three different proportions of cement, sand, and broken stone, and three different quantities of water were used. These specimens were made during the first two weeks of January, 1909, and were immersed in the sea at the Charlestown Navy Yard, in the last week of February, 1909.

They were suspended so that more than 3 ft. of each specimen at the top would seldom be wet and, presumably, never be immersed, whereas about 3½ ft. projected below mean low water, and, therefore, would never be uncovered.

The details of the making, the progress, and the reports of the condition of these specimens have been described in two pamphlets entitled, "Aberthaw Tests on Concrete in Sea Water", issued in 1914 and 1920, respectively, in a number of articles in the technical press, and most fully by R. E. Bakenhus, M. Am. Soc. C. E.† Reference is made to these sources for full details. Only such extracts from those data will be given as are necessary to make this paper intelligible.

After 13¼ years of study of these specimens, the writer believes it is possible to make some tentative deductions on the cause of failure and suggestions for design so as to make future structures as nearly as possible proof against disintegration by sea water.

The conditions in Boston Harbor are probably as severe as in any harbor of the United States. The specimens are subjected to considerable tide run, attacked by waves, abrasion from floating objects, freezing during severe winters, and attack from impurities, chemical or otherwise, present in sea water.

In April, 1914, the Navy Department put into commission fuel-oil pipes adjoining these specimens. Oil spilled on the sea has heavily coated them from near their tops to low tide. The writer believes this coating retards the action by the sea water and, therefore, will lengthen the time required for this test, which, however, will not destroy its value.

At low tide, on May 22, 1922, the writer made an examination of the specimens. No appreciable change could be observed from the condition as

* Pres., Aberthaw Constr. Co., Boston, Mass.

† *Transactions*, Am. Soc. C. E., Vol. LXXXI (December, 1917), p. 645.

described in the 1920 report of the Aberthaw Company, and the order of deterioration was the same as that given by Capt. Bakenhus in his report of 1917.

Two of the twenty-four original specimens, Nos. 19 and 21, have been lost; and five are in poor condition and may be considered to have failed. These five, which are Nos. 7, 11, 13, 22, and 23, were made of 1:3:6 concrete. Capt. Bakenhus makes nine classes, or degrees, of failure, placing No. 7 in the worst class, Nos. 22 and 23 next to the worst, that is, in Class 8, and Nos. 11 and 13 in Class 6, fair condition.

Specimen No. 7 was mixed dry, whereas the other specimens were mixed wet. Specimens Nos. 1, 4, 8, and 9 show some deterioration. Nos. 1 and 4 are of rich proportions, but mixed dry, whereas, Nos. 8 and 9 are of plastic and wet mixtures, but of lean proportion. The remainder, Nos. 2, 3, 5, 6, 10, 12, 14, 15, 16, 17, 18, 20, and 24 are in fairly good condition.

On May 24, 1922, Specimens Nos. 7, 11, 22, and 23 shown on Fig. 8, were removed for testing. No. 7 was made of three average, normal Portland cements, Vulcanite, Alpha, and Giant, in equal parts, thoroughly mixed, and then repacked in bags for use in mixing the concrete. No. 11 was made of Blanc (white Portland cement), very wet. No. 22 was made of 90% of the same cement as No. 7 and 10% of hydrated lime, very wet. No. 23 was made of cement like that of No. 7, plus Sylvester solution of soap and alum, very wet.

When the specimens were removed from the water they had, at the tops, about ¼ in. of gummy, sticky adhesion of fuel oil and dirt, which, toward the low-water mark, became thinner with a harder, somewhat glossy, surface.

No oil could be noted below the low water-line, but the specimens were coated with live mussels, which indicated that the impurities of the water were chiefly on the surface and were not sufficient to kill marine life.

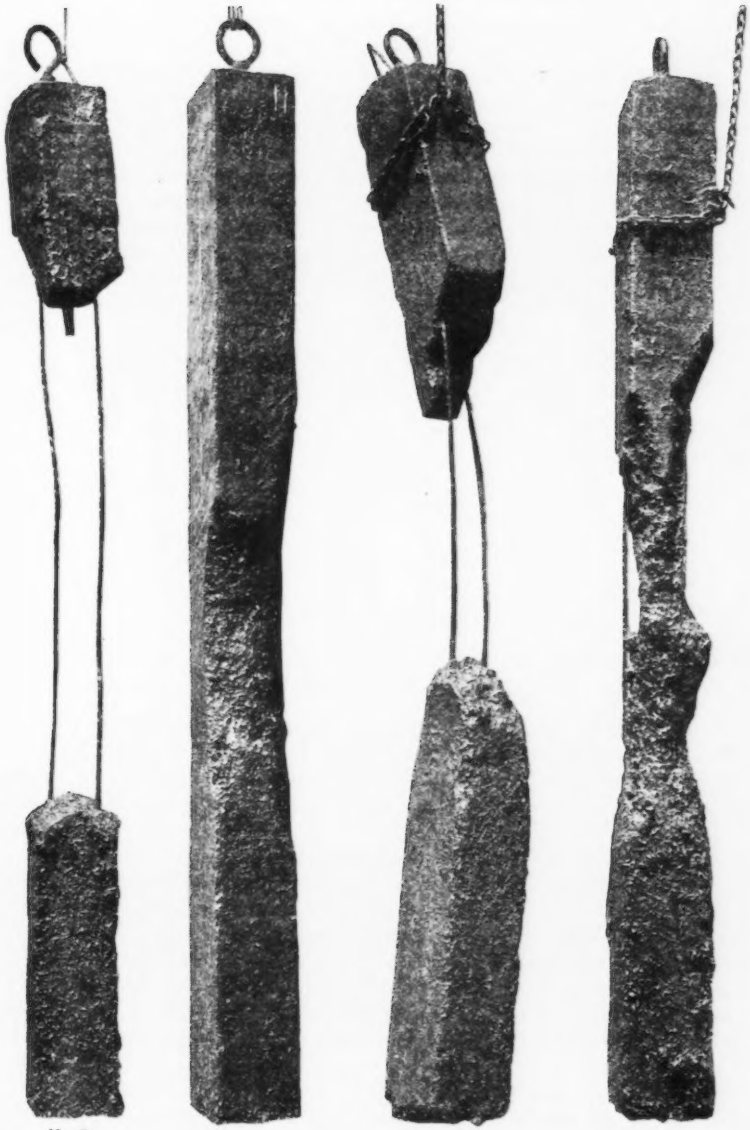
Some of the specimens had previously been cracked in being removed from the water and some pieces that broke off at the cracks showed a slight brownish discoloration, which indicated that the oil had not penetrated very deeply or in any volume into the cracks. No evidence of penetration of oil below the surface of even the poorest specimens could be detected at fresh fractures of the concrete. Discoloration was not more than 3/32 in. deep. In all these specimens, the parts most damaged, namely, those near mid-tide, showed a rather disintegrated and soft concrete.

The specimens were cut to approximately square faces, the length of the test pieces being more than twice the width of the cross-section. The steel reinforcement was cut off and burned below the surface. The test pieces were then faced to a flat surface resembling terrazzo.

The top end of Specimen No. 7 was so short and so badly cracked that it was impossible to obtain a test piece larger than about 8 by 8 in. by 15½ in. All the others were of the full cross-section and about 3 ft. long.

Through the courtesy of H. W. Hayward, M. Am. Soc. C. E., they were tested at the Massachusetts Institute of Technology on June 7, 1922.

A piece of blotting paper was placed on top of the first specimen and under both ends of the remaining specimens. From the impression made on



No. 7. No. 11. No. 22. No. 23.

FIG. 8.—TEST SPECIMENS.

these sheets, it was possible to obtain the net bearing area and the character of the bearing. The edges of the test pieces were irregular and, on some, the corners were broken so that it would have been difficult to obtain the bearing area by direct measurement. In Table 8 is given the average bearing area of each end, which area is approximate only, and the writer is convinced that it is too large, rather than too small. The specimens were tested only to the point of failure. Cracks appeared longitudinally, and formed roughly two prisms the shape of an hour-glass. Reinforcing steel that was exposed after the tests, appeared to be bright. Specimen No. 7, the most porous one, was so broken after the test, that the reinforcing bar was removed. With the exception of three spots about ¼ in. in diameter which showed rust, the bar was entirely bright. This bar was from the lower end of the specimen which had been immersed for 13¼ years.

TABLE 8.—Actual Bearing Area of Test Specimen.

No. of piece.	Bearing.	Area, in square inches.	Average area, in square inches.
7 Top	Top	51.8	
	Bottom.	62.8	57.3
7 Bottom	Top.	232	
	Bottom.	230	231
11 Top	Top.	236	.....
11 Bottom	Top.	240	.....
22 Top	Top, poor.	200	.....
	Bottom.	215	207.5
22 Bottom	Top.	217	
	Bottom.	208	212.5
23 Top	Top, fair.	213	
	Bottom.	238	226.5
23 Bottom	Top.	221	
	Bottom.	224	222.5

For chemical analysis, a sample was taken from the top of the specimens, one from the lower corner of the bottom specimen and one from near the center. Mr. Herbert L. Sherman, a cement chemist, co-operated with the writer in making chemical and physical analyses of the cement used in the specimens.

After a careful and complete analysis of three samples each from Specimens Nos. 7 and 23, and a thorough study of the results, Mr. Sherman reports that it is impossible to draw any conclusions.

The result of the crushing test is given in Table 9. The original cross-section is copied from the earlier report, and is not a scaling of the specimens at the present time. It will be noted that the normal mixture of concrete shows results as good as could be expected of any concrete of this mixture and age. It will be noted also that the lower end of each of the four specimens was stronger than the upper end.

Omitting Specimen No. 7, from which only a small sample could be tested and which, therefore, may not be directly comparable with its own bottom, the other three show the strength of the top to be about 92% of that of the bottom. This indicates that there is a common law as to the condition of the concrete when permanently wet as compared with dry.

TABLE 9.—CRUSHING LOAD ON SEA-WATER TEST SPECIMEN.

No. of piece.	Original cross-section, in inches.	Area, in square feet.	Length of test specimen, in inches.	Actual section, in inches.	Total load, in pounds.	Load, in square inches, actual section.	Ratio of strength, top to bottom.
7 Top.....	15⅞ by 16 3/16	1.82	15½	57.3	157 300	2 750	0.82
7 Bottom....	15⅞ by 16 3/16	1.82	35½	231	778 800	3 375	
11 Top.....	16 by 16¾	1.86	36	236	750 200	3 180	0.93
11 Bottom....	16 by 16¾	1.86	37	240	823 900	3 435	
22 Top.....	15⅞ by 16¾	1.80	30¾	207.5	463 100	2 235	0.92
22 Bottom....	15⅞ by 16¾	1.80	36	212.5	562 100	2 645	
23 Top.....	16 3/16 by 16¼	1.82	32½	226.5	391 600	1 730	0.91
23 Bottom....	16 3/16 by 16¼	1.82	33¾	222.5	424 600	1 907	

The following summary of tests on 1:3:6 concrete mixed dry, plastic, and wet, on four brands of cement, cured in air for 1.6 to 1.75 years, is given for comparison with laboratory specimens. The data on these specimens, which were 12-in. cubes, were taken from the Annual Report of the State Engineer of New York for the fiscal year ending September 30, 1897. Fifty-two specimens were crushed at the Watertown Arsenal, the dry specimens being 9% stronger than those from plastic or wet concrete:

Highest crushing strength per square inch.....2 887 lb.
Lowest " " " " " .....1 716 lb.
Average " " " " " .....2 095 lb.

In 1912, the writer built, in Portland Harbor, a fish pier that consists of a wharf, supported by concrete piers about 4 ft. square, with a heavy floor forming the wharf platform, on which a fish house was built. Most of these piers were built inside a coffer-dam, and, therefore, were cast dry and allowed to set more than a month before being subjected to the tidal action of the sea.

The two rows of piers nearest the shore, being out of water except during high tide and never being immersed more than about 3 ft., were cast between tides. An examination of these piers was made on May 25, 1922, at which time they were ten years old. All the piers cast within the coffer-dam appeared to be in first-class condition, whereas those cast between tides showed some signs of deterioration below the high-tide line.

In 1902, a series of concrete specimens was made, in order to test the protection of steel from corrosion. The concrete was of 1:3:6 proportion, with the expectation that it would be porous. The blocks were 4 in. square and 12 in. long, and had a ½-in., square twisted steel bar, 16 in. long, through the center, that projected at both ends.

At Fort Warren, the War Department immersed one series of these specimens in sea water and, after three years' testing, the remainder of this series was lost.

The second series was given to the Chief Engineer of the Metropolitan Sewerage Commission, and the specimens were immersed in a trunk sewer. After seven years, the remainder of this series was lost.

The third series of specimens was buried in ground that was always damp and at times was saturated with water. After being twenty years in the

ground a specimen from this series was removed and split open. The steel was bright in every specimen of all three series where the cement was in close contact with it; rust was present where there were voids. When the bars were originally embedded, they were coated with hard rust, but no scales. These results indicate that cement will protect steel from corrosion in unfavorable conditions.

Although there has not been time to draw final conclusions from these series of tests, and although the series is not sufficiently comprehensive to prove a general rule for all cases of immersion of concrete in sea water, it is thought that some conclusions are now justified, and ought to be helpful to those designing marine structures.

It appears, first, that the mechanical action of the elements is much more vital than the chemical action. The densest specimens show the least wear. The porous specimens which were built with the expectation that they would soon disintegrate, have done so.

Chemical action must be considered. The specimens in which the cement was low in alumina, both of lean and of rich mixtures, are in good condition. Those that had foreign ingredients (Nos. 22 and 23), of hydrated lime and Sylvester solution, show serious weakness, whereas one specimen containing 5% of clay as filler to make the concrete dense, is still in quite good condition.

The question of the quantity of water used in mixing must be considered. In a paper* by Mr. Dan Patch, photographs are shown of the specimens arranged in three horizontal rows, in the order of the quantity of water used in mixing, and superimposed is a curve showing the resistance to compression and to wear as determined by the Research Laboratory of the Lewis Institute of Chicago, Ill. The condition of the specimens shown in these photographs indicates that those containing from 9 to 10% of water are in better physical condition than the very dry or the very wet specimens.

If good workmanship is observed in the placing of concrete in forms, excess of cement and mortar is brought to the surface and produces a skin which, until abrasion has removed it, resists very well the action of sea water. The concrete is not as durable in resisting wear as this skin.

Specimens made of the poorer materials, which fail the earliest in their lower portions, due to tidal action, have proved to be durable if permanently immersed. An explanation of the reason for the upper end of the specimen showing less strength than the lower end may be due in part to the matter of suspension. In order to handle the specimens readily, $\frac{3}{4}$-in. square twisted steel bars were extended about 2 in. from the surface the full length at opposite corners. In the center of the specimen, a hole was cored into which was grouted a hook with a shank about 3 ft. long, by which the specimens were suspended. The weight of the specimen produces a tension of about 100 lb. per lin. in. in the upper 3 ft. This condition may have produced porosity, or invisible cracks, through which weathering took place. The upper end was subjected to yearly variations of temperature of probably 100° Fahr.—sometimes, to changes of many degrees within a few hours—at times to a film of ice on the surface, and to a slight degree of abrasion of floating objects in the water.

* *Engineering News-Record*, March 3, 1921.

It seems safe to conclude that if rich concrete is made from a good quality of cement, of normal composition, or low in alumina, and the material is thoroughly mixed, using 9 to 10% of water, and placed in the dry with careful spading to assure a dense surface, that durability in sea water can be assured unless the specimen is subjected to considerable abrasion, in which case it would be wise to face the concrete with a protective coating within the limits of the rise and fall of the tide. Below low tide, and above high tide, concrete as described is a practical, satisfactory, and proper building material.

DISCUSSION

HENRY S. ADAMS,* M. AM. SOC. C. E.—In his paper, the late Mr. Hodgdon described much of the new marine work that has been done around Boston Harbor, and the speaker has had a great deal to do with some of the old work that has been built there. At first, the principal kind of construction was timber cribs filled with stone. The first record that the speaker has been able to find, Boston having been settled in 1632, was that of September 10, 1673, when the town voted to "Errect a Wall or Wharfe upon the flats before the Town, from the sconce to Capt. Scarlet's Wharfe, or useing some other meanes for secureing ye said Town from Fire Ships in case of the approach of an enemie." This wall or wharf was described as being about 2 200 ft. in length, 22 ft. in breadth at the bottom, to be raised from 14 to 15 ft. in height, and to be 20 ft. broad on top for guns, and to be constructed of wood and stone. This barricade, or out wharf, as it was called, was built, and, in 1872, Atlantic Avenue was constructed over it. There is an old affidavit to the effect that it was built as a crib.

In 1710, Long Wharf was built. It is described in some of the old books as one of the longest wharves in the world, its length being more than ½ mile, reaching from Chatham Row to low water. It was built in the form of a crib. The speaker has had occasion to pull up parts of that wharf, and the timber is as sound at present, apparently, as the day it went in. It was built of white pine.

"T" Wharf, which was an excrescence on the side of Long Wharf, was built in 1715. Part of that wharf was removed for placing the foundations for the Quincy Market Cold Storage, and the timber was found to be sound, in fact, there was some difficulty in removing it.

Long Wharf, in Portland, Me., was built in 1797 of crib work. When this wharf was removed in order to build another structure, a large proportion of the logs were found to be of white pine, and trouble was encountered because the timber floated. Logs from 20 to 24 in. in diameter were dug up, that were about 20 ft. below low water. The use of piles in the building of wharves began about eighty years ago. The speaker has had occasion to pull down some of those wharves, and evidences of teredo action have been found. The teredo is active in Boston Harbor as far as Lewis Wharf. Some of the timbers were full of them. The teredo and the allied limnoria are found as far as Constitution Wharf and the piles are badly eaten. Crib wharf construction has largely ceased since pile construction began, that is, south of Maine.

FREDERIC H. FAY,† M. Am. Soc. C. E.—Of the many harbors along the coast of Maine, the one of prime importance is that at Portland, where not only the chief seaport of Maine has been developed, but also an important winter port for Canada.

Portland Harbor.—Portland, located at the southwesterly end of Casco Bay, has one of the finest natural harbors in the world. It is almost on the

* Cons. Engr., Boston, Mass.

† Cons. Engr. (Fay, Spofford and Thorndike), Boston, Mass.

open sea, but still is well sheltered; it has ample space for anchorage basins and a natural deep-water channel from the open sea almost to the shore line of its water-front.

The principal piers are within 3½ miles of the open ocean and the approach channel is so broad and straight that steamships calling regularly at this port frequently dispense with pilots and enter or leave the harbor at any time under the direction of their own officers.

Along the water-front of the city opposite that part devoted to overseas commerce is a dredged channel, 35 ft. deep at mean low water, and 30 ft. deep for the remaining distance up Fore River. The character of the harbor bottom is such that this water-front channel may readily be deepened whenever conditions warrant. The dredged parts of the harbor require little maintenance, because of the almost entire absence of sediment in its waters. The mean tidal range at Portland is 8.9 ft. and there are no troublesome tidal currents.

Although Portland is the most northerly of the principal Atlantic ports of the United States, as well as the port nearest Europe, its harbor is notably free from ice and its water-borne commerce is not interrupted throughout the severest winter.

The strategic importance of this port has been recognized by the Federal Government and Portland has been made one of the most strongly fortified harbors in the United States.

Railroads Entering Portland.—Portland is served by three steam railroads: The Boston and Maine, which operates three lines into the city; the Maine Central with its three main lines; and the Grand Trunk Railway of Canada.

The Boston and Maine Railroad connects Portland with the important manufacturing centers of New Hampshire, Vermont, and Northern Massachusetts.

The Maine Central Railroad, by its own lines or by its connection with the Bangor and Aroostook Railroad, covers practically the whole State of Maine. The Maine Central's line through the White Mountains makes connections with the Boston and Maine, Canadian Pacific, and Quebec Central. Through its connection with the Canadian Pacific Railway, the Maine Central furnishes a route to Montreal, Que., Canada, Detroit, Mich., Chicago, Ill., the Great Lakes, and the greater part of Canada, especially the Canadian West.

The principal Atlantic terminus of the Grand Trunk Railway of Canada is at Portland and this port alone is used for its winter shipping for overseas traffic. The Grand Trunk affords a direct route to Montreal, Buffalo, N. Y., Detroit, Toledo, Ohio, Chicago, and the Great Lakes, and connects with the Grand Trunk Pacific Railway crossing the Canadian West. Grain is the principal commodity exported over the Grand Trunk docks at Portland, nearly 43 000 000 bushels having been handled in the season of 1915-16. This is equal to the total grain shipments from the Port of New York for 1921.

During the winter of 1921-22, 17 000 000 bushels of grain were shipped through the Portland Terminal of the Grand Trunk, and 99 trans-Atlantic and 37 coastwise vessels have berthed at its piers.

Portland the Natural Winter Port for Canada.—Portland is pre-eminently the natural winter port for the Dominion of Canada. It is only 297 miles from Montreal, and its connection with that city, *via* the Grand Trunk affords a rail route of relatively easy grades, particularly favorable for the movement of east-bound freight. The Canadian winter ports of St. John and Halifax, on the other hand, are, respectively, 481 and 842 miles from Montreal, *via* routes of steeper grade over which freight movement, especially in winter, is much more difficult and expensive. Had Maine been Canadian territory, Portland would no doubt be the leading port of Canada, or, at least, equal in importance to Montreal.

Portland's Belt Line Railroad.—Portland possesses one asset of great importance to the development of any port, namely, a belt line railroad which reaches every part of its water-front. This belt line is made up of trackage of the Portland Terminal Company (which is owned jointly by the Boston and Maine and the Maine Central Railroads and handles the traffic of both these roads in and near Portland) and of trackage owned by the Grand Trunk Railway. Along the important Commercial Street water-front of the city, each company has running rights over the tracks of the other, so that any wharf in this district is accessible on equal terms to all railroads entering the city.

Portland's Water-Front Developments.—Portland is well supplied with coal-handling and fuel-oil plants, most of which are up to date in capacity, equipment, and depth of water at adjoining berths. It also has suitable wharves for handling pulp wood, sulphur, and china clay used in the paper industry, not only of Maine, but of other parts of the country.

The remaining wharves along the water-front, with the exception of the Grand Trunk Terminal, are old and more or less obsolete, having been built in the days of sailing ships, when Portland had an extensive trade with the West Indies. These wharves are now used either for coastwise shipping, or by business and manufacturing concerns, many of which could be accommodated with equal facility away from the water-front.

Grand Trunk Railway Terminal at Portland.—Until the present time, the only overseas terminal at Portland has been that of the Grand Trunk Railway System, comprising New, Ocean, Atlantic, and Galt Wharves, with two grain elevators with a combined capacity of 2 500 000 bushels and the supporting railroad yard. The first three of these wharves have transfer sheds and are used for overseas business. Although not of the latest type of pier construction, these wharves with their sheds are maintained in good condition and during the five winter months when navigation of the St. Lawrence River is blocked by ice, they accommodate a large volume of business with reasonable efficiency and dispatch. Until recently, harbor-line restrictions limited the length of these wharves to about 500 ft., a length inadequate for the larger vessels now using this terminal. In 1920, the War Department established a new harbor line about 500 ft. out from the old one, making possible the

extension of these piers so that in the future they may have a total length of 1 000 ft.

Portland's Need of Further Water-Front Development.—Within the last few years, the need of increased water-front terminal facilities has been keenly felt at Portland. Certain steamship lines have sought to establish themselves at this port, but have been unable to do so because of the lack of facilities for the accommodation of their ships. Others already established have found it difficult at times to secure suitable berthing space. The Canadian Pacific Railroad is understood to have recently sought entrance to Portland over the Maine Central Railroad and if suitable water-front terminal facilities were available, some Canadian Pacific business would probably pass through this port. The Maine Central Railroad, however, like most other roads, has not been in condition financially to embark on further water-front improvements.

Public Development of the Portland Water-Front.—Recently, however, through the instigation of the Portland Chamber of Commerce, the need of further port development at Portland has been forcefully impressed on the people of that city and of the State at large. It is now recognized that port development should not be left entirely to railroads or other private interests, as has been the case at Portland heretofore, but should be, at least in part, under public control, not only to insure development on the broadest possible lines to the greatest benefit to the State and the city, but because, in addition to the direct benefits accruing to rail and water carriers, the community at large derives a substantial indirect benefit from such developments. In the City of Portland, several millions of dollars are spent annually by rail and water carriers in the purchase of coal, provisions, and other supplies from local dealers and for the wages of longshoremen, freight handlers, and others. It is recognized that for every ton of local freight shipped to and from the city there are many tons shipped to and from the district immediately tributary to the port, as well as the inland territory of the country. The people of Maine believe that with further development of the Port of Portland, making possible an increase in coastwise as well as overseas business, the industrial and agricultural development of the entire State will be stimulated, especially as the State has available much undeveloped water power which, in these days of high priced coal, may now be economically utilized to provide cheap fuel for industrial uses.

Through legislation passed in 1919, the State and the Cities of Portland and South Portland have jointly embarked on a policy of further development of the Port of Portland at public expense. The two cities have provided the site of the first of a series of publicly owned piers and the State is constructing such a pier alongside the Grand Trunk Terminals.

New State Pier at Portland.—The site for the new State Pier, which has been purchased and deeded to the State by the Cities of Portland and South Portland acting jointly through the State Pier Site District recently established by legislative enactment, consists of Franklin Wharf, formerly owned by the Eastern Steamship Lines, Incorporated, and the berthing place of that company's coastwise lines, and of Galt Wharf, which was formerly owned by

the Grand Trunk Railway. The State is now constructing on this site a modern pier 1 000 ft. long, to be equipped with adequate sheds and modern freight-handling equipment. For the present, the pier is designed to be used both for overseas and for certain coastwise business, and it will be adequate to accommodate the largest ships that will come to Portland for many years. The new pier will project nearly 500 ft. beyond the existing piers of the adjacent Grand Trunk Terminal and will extend to the newly established pier-head line previously mentioned.

In order to provide a slip of suitable width and depth between the Atlantic Wharf of the Grand Trunk Railway and the new State Pier, Galt Wharf, which was partly an old pile structure and partly a solid fill, has been completely removed, thus providing a slip 250 ft. wide with a depth of 35 ft. alongside the new State Pier. Ultimately, this slip may be deepened to 40 ft. if desired.

In the initial development of the State Pier, the existing Franklin Wharf is being utilized in conjunction with the new work, and the new pier is being built for a width less than that proposed for its ultimate development. This will afford a berthing space of 1 000 ft. along the easterly side of the pier, which will be used for overseas and Pacific Coast traffic. The westerly side of Franklin Wharf, as at present, will continue to be used by the Eastern Steamship Lines, Incorporated, for coastwise traffic, and that company is about to re-establish a freight line to New York City, which will be berthed at the westerly side of the new pier at the outer end of Franklin Wharf. Later, when conditions warrant, it is proposed to rebuild Franklin Wharf and to widen the State Pier so as to provide a complete new structure 1 000 ft. long, 320 ft. wide, with a least depth of 35 ft. of water on all sides.

The new pier will consist of creosoted Southern pine piles supporting a deck of reinforced concrete at the inner end and a timber deck elsewhere. The pier will have two transit sheds on its easterly side. The shed at the inner end is to be 90 ft. wide by two stories high and is designed primarily for trans-Atlantic shipping with accommodations for freight on the lower or main deck and quarters for saloon passengers and immigrants on the second floor. The shed at the outer end will be 60 ft. wide and one story high and designed for freight service only. On the westerly side of the new pier, a shed 60 ft. wide is now being erected for the New York freight service. At the easterly side, the cap-sill will be 27 ft. wide between the sheds and the fender cap, with a railroad track providing for the operation of a locomotive crane and with provision also for the installation later of semi-portal gantry cranes. Railroad tracks and trucking driveways are provided also in the middle of the pier.

Grain will be brought to the State Pier from the elevators of the Grand Trunk Railway Company by an extension to the grain conveyor system of that company through a modern fireproof gallery to be constructed by the State along the easterly side of the pier, the gallery being of sufficient size for the installation of a second belt in case it should be found desirable to provide for the handling of grain from other railroads than the Grand Trunk. A working agreement has already been made between the State and the Grand

Trunk Railway by which grain will be delivered to ships at the State Pier on the same terms and conditions that it is delivered to ships at the Grand Trunk piers.

Portland Port Authority.—The administration of this new publicly owned terminal is in the hands of a State Board, known as the Directors of the Port of Portland. The Board, created by legislation passed in 1919, is composed of five members, one of whom represents the City of Portland and was appointed by the Mayor of that City, and the others represent the four Congressional Districts of the State and were appointed by the Governor. This Board is the administrative agency of the port and on it is conferred the power to acquire property by eminent domain, to make all necessary plans for the comprehensive development of the port, to take charge of and construct piers and other public works on State property, to operate such piers and administer all public works under their control, and to keep thoroughly informed as to the present and probable future needs of the port.

WILDURR WILLING,* ESQ.—Federal river and harbor works in the New England States from the Canadian border to Chatham, Mass., on the southeast point of Cape Cod, are in charge of the District Engineer of the Boston, Mass., District, a recent consolidation of the former Portland, Me., and Boston, Mass., Districts.

The following are the principal works included in the District: St. Croix River, Bar Harbor, Penobscot River, Rockland Harbor, Georges River (Thomaston Harbor), Portland Harbor, and Saco River, Maine; Newburyport Harbor, Merrimack River, Harbor of Refuge at Sandy Bay, Cape Ann, Boston Harbor, Gloucester Harbor, Beverly Harbor, Salem Harbor, Lynn Harbor, Mystic River, Malden River, Dorchester Bay and Neponset River, Weymouth Fore River, Weymouth Back River, Plymouth Harbor, and Provincetown Harbor, Massachusetts.

The principal work has been the deepening of channels and anchorage basins by dredging and removal of ledge rock; the creation of artificial harbors by the construction of breakwaters; the deepening of bars at the mouths of certain rivers by jetties; and the preservation of shore lines.

Shore protection works on the part of the United States Government date back to 1824, and with peculiar historical fitness were first undertaken at Plymouth, Mass., for the protection of Long Beach.

Long Beach.—The existence of Plymouth Harbor depends entirely on the protection and preservation of Long Beach, which is a narrow strip of land that extends 2¾ miles out from the mainland in a northwesterly direction, nearly parallel to the shore of the Town of Plymouth and distant from it about 1 mile. It affords to the harbor its only shelter from easterly storms. For the protection and preservation of this beach, various works have been built from time to time, until, finally, they have proved efficient and successful.

The works originally undertaken for the maintenance of Long Beach were light temporary structures, of various designs, which may be described as follows: The bulkheads consisted of triangular frame or of cribwork of tim-

* Lt.-Col., Corps of Engrs., U. S. A.; Dist. Engr., Boston, Mass.

ber placed at intervals of 4 ft., covered with 2-in. plank, set in a trench 12 ft. wide and 3 ft. deep, the trench and frame being filled to the level of the ground.

The jetties consisted of a framework filled with brush and ballasted with stone. The groynes consisted of small stones intermixed with brush, and sometimes brush alone. They were built perpendicularly from the main bulkhead on the seaward side, spaced 50 to 150 ft. apart, and varied in length from 75 to 265 ft., aggregating 5 250 ft. Others have been built inside the bulkhead in places where most necessary. They were made by placing brush on the surface of the beach and piling beach boulders on it to a height of about 2 ft. for a width of 5 ft. The brush groynes were made by digging a trench 2 ft. wide by 2 ft. deep, and placing brush in it upright and as close as practicable. The brush is held in place by sand well rammed into the trench, and sometimes further protected by small boulders placed in a row along the sides of the groyne. The brush rises 2 or 3 ft. above the surface of the beach.

On account of the light design of these structures, repairs had to be made from time to time until finally rubble stone dikes were built, and they have fully served their purpose and continue in good order to the present time.

Boston Harbor.—The first sea-wall was started in 1825, to preserve Georges Island, an important site for one of the defenses of Boston Harbor. Sea-walls have been constructed at Deer Island, Castle Island, Rainsford Island, Lovells Island, Great Brewster Island, Long Island, Gallops Island, and Point Allerton. All these walls have been built by the Federal Government under the direction of the Corps of Engineers. From surveys made in 1820 and 1840, respectively, it appeared that 5.7 acres had been washed away from Great Brewster Island in the interval. This is about one-fourth of the whole island, at the latter period. The 1840 survey disclosed that the preservation of the islands from the wash of the sea was indispensable, as covers of the anchorages and roadsteads, and also to the maintenance of requisite depths in the channels. The aggregate length of these walls is about 3¾ miles, and the total amount expended in their construction and maintenance is about $1 500 000.

In his paper, the late Mr. Hodgdon gave an excellent description of the three main ship channels leading into Boston Harbor. The total of all Federal expenditures to June 30, 1921, on these and the various subsidiary channels of Boston Harbor, including Chelsea Creek, Fort Point Channel, and the Mystic and Malden Rivers, Dorchester Bay and Neponset River, and Weymouth Fore and Back Rivers, has been $12 734 167 for new work, and $948 584 for maintenance. The total commerce in 1921 was 9 752 841 tons. On a capitalization of 4% and an estimate of $40 000 per year for maintenance, there results a charge to the United States of 5.6 cents per ton.

Portland Harbor.—The second harbor in importance on the New England Coast is that of Portland, Me. The existing project for Portland Harbor provides for a commodious anchorage off the eastern end of the city, with a minimum depth of 30 ft.; for the dredging of the greater part of the inner harbor to a depth of 30 ft., except for the lower part of the main harbor and its channel of approach, where the depth is to be 35 ft.; for a channel 30 ft.

deep and 300 ft. wide in Fore River up to the upper Boston and Maine Railroad Bridge; for a channel 30 ft. deep and 300 ft. wide from the anchorage to the Grand Trunk Bridge at the mouth of Back Cove; for rock excavation to give a depth of 14 ft. between the Grand Trunk and Tukeys Bridges, and a channel 12 ft. deep and 300 ft. wide thence to the head of Back Cove; for the removal of two obstructing ledges in the main ship channel to a depth of 40 ft.; and for a stone breakwater about 1 900 ft. long on the southerly side of the mouth of the inner harbor. The project depths refer to mean low water. The mean tidal range is 8.9 ft. and the extreme, 10.2 ft., although variations as great as 16 ft. have been observed under storm conditions.

The existing project executed by dipper and clam-shell dredges in material similar to that in Boston Harbor has been completed, except for the removal of ledges in the main ship channel, estimated to cost $100 000. The expenditures to June 30, 1921, have been $2 271 386, and the annual maintenance is estimated at $3 000. The total commerce in 1921 was 2 369 881 tons. This results in a cost of 4 cents per ton.

Important breakwaters have been constructed at Rockland, Portland, and Bar Harbor, Me., and at Gloucester, Mass.

Rockland Breakwater.—The Rockland Breakwater extends from Jameson Point 4 350 ft. in a southerly direction. It consists of a mound of rubble with side slopes of 1:1 to a grade of 10 ft. above mean low water, at which grade the breakwater is 20 ft. wide. On this mound is a capping course of heavy split stone granite 15 ft. wide, bringing the top of the breakwater to a grade of 14 ft. above mean low water. The total cost of the breakwater was $615 000, and the annual maintenance is about $500.

The breakwater was completed in 1904, and stood undamaged until the winter of 1920-21, when some of the rubble mound on the seaward side slipped downward, the repairs for which are estimated to be about $9 000.

Bar Harbor Breakwater.—There is also a breakwater at Bar Harbor, extending from Porcupine Island to Dry Ledge and thence in a straight line to within 600 ft. of Mt. Desert Island, to protect the harbor from southerly storms. The breakwater was under construction when the World War broke out and the work was suspended on account of high prices and scarcity of labor. It has been estimated that about 15 000 tons will be required to bring it up to the grade of mean high water, and bids have been asked recently for the completion of the job.

Gloucester Breakwater.—The Gloucester Breakwater extends from Eastern Point over Dog Bar, 2 250 ft. in a northwest direction. It consists of a mound of rubble 31 ft. wide at mean low water, surmounted by a superstructure extending 17 ft. above the grade of mean low water, formed by two dry walls of heavy split stone, inclosing a core of rubble, capped by heavy stones forming a top course 10 ft. wide, the slopes of the rubble structure being, on the harbor side, 1:1.3, on the seaward side 1:3 to a grade of 12 ft. below mean low water, and 1:1.5 thence to the bottom. On the seaward side, the breakwater has an apron of heavy rubble rising to approximately 6 ft. above mean low water, the object being to protect the lower course of stone in the super-

structure. The total cost was $410 097 and the annual maintenance is about $1 000.

The Sandy Bay Breakwater project has been recommended to Congress for abandonment. Only recently the remaining small balance in the hands of the District Engineer was diverted by authority of Congress to emergency use on the 1922 floods of the Mississippi River.

R. S. PATTON,* M. AM. SOC. C. E. (by letter).—The problem involved in the design and location of engineering works intended to protect the shore from erosion by the sea is, in many cases, a peculiarly perplexing one. That problem involves, on the one hand, attack by a combination of forces each of which varies greatly from time to time both in its strength and in its direction; and, on the other hand, a resistance to attack determined by local factors of which the configuration of the shores, the character of the material composing them, and their accessibility to attack are merely typical.

This means that the engineer undertaking the erection of protective works must consider each locality as a unit. No adequate general rules can be laid down for his guidance. For example, studies of conditions along the New Jersey Coast indicate (the studies have not progressed sufficiently to justify a final conclusion) that each of the many inlets along that coast presents its own problem, and that protective works which would accomplish their purpose at one inlet might fall short of such accomplishment at the adjacent inlet, 10 miles distant. The writer does not mean to imply that there is any radical variation, in that short distance, in the nature of the attack from seaward. It seems more probable that the varying results noted are due to modifications of the seaward attack by the body of inland water.

It follows, therefore, that the confidence with which the engineer can undertake the construction of protective works will be in direct ratio to the thoroughness with which it has been practicable for him to determine the nature of the change which he undertakes to check, and the character of the forces producing it. Unfortunately, neither of these factors can be readily determined. In the United States, data on the subject of shore protection are meager, and such data as exist are not in a form readily accessible to the engineer. Consequently, many forms of protective works have been erected, which have failed to meet expectations, largely because local or private interests have not been in a position to undertake the extensive research necessary to ascertain the facts which, under more favorable conditions, would have formed the basis for the design of their structures.

The Federal Government has collected much data bearing on this subject. In practically all cases, this material has been gathered to serve some specific purpose other than shore protection, and its additional value in connection with studies of this character has been overlooked or ignored. It is fragmentary; one Bureau has collected a certain set of facts, a second Bureau another set. Each, by itself, may be of little value for this purpose; combined, they may be of great value. It is to be hoped that some day this material will be placed in shape for the use of the engineer. Meanwhile, however, the engi-

* Hydrographic and Geodetic Engr., U. S. Coast and Geodetic Survey, Washington, D. C.

neer derives little benefit from it because, in most cases, he either does not know of its existence, or he does not know how to secure it.

It is not practicable for the engineer to obtain and utilize all this information. After eliminating a large part of it, a remainder exists, in most cases, readily obtainable at slight cost, which bears so directly on his problem that he would make a great mistake if he failed to take advantage of it if the project is of magnitude. The writer will endeavor to indicate briefly the character of this latter assistance which it is possible for various agencies of the Federal Government to render.

The engineer must seek answers to two questions: (1) What is the nature of the change taking place; and (2), what forces are causing it?

Much progress toward the answer to the first question can usually be made by studying the topographic and hydrographic surveys made by the Federal Government. For the past hundred years, the Coast and Geodetic Survey has been making such surveys along the Atlantic Coast, and the frequency of re-surveys usually depends on the rapidity with which changes are occurring. For example, the New Jersey Coast is covered by three series of topographic surveys and an additional one by aerial photography, with from six to ten surveys of the various inlets. The Army Engineers, also, have made many surveys at localities where improvements have been proposed.

The surveys by these two organizations furnish a picture of successive stages in the development of the present shore, sufficiently separated to eliminate effects due to transitory causes, and thus enable one to deduce with considerable accuracy the general character of the evolution in progress.

The next step is to discover what forces operate and how they operate, to cause the changes observed. This is a much more difficult question than the preceding one, and the engineer can nowhere find the answer as he did in the first case. The best he can do is to secure data from all possible sources and then rely on his own judgment for the proper correlation of such data with the changes observed.

A great deal of valuable information can be obtained from persons having local knowledge, but information of this kind must be used with caution. It is usually fragmentary and superficial. Temporary effects and apparent causes are mistaken for basic factors. Therefore, the assurance with which such material can be utilized will be increased if the engineer can secure a background of fact against which to measure it. The material which the Federal Service can furnish, is valuable as supplying this background.

The Weather Bureau has detailed statistics from which may be determined wind effects, the direction, frequency, duration, and velocity of prevailing winds and storm winds, respectively.

The Coast and Geodetic Survey can furnish detailed information regarding the tides and currents at many points along the coast.

The Bureau of Lighthouses and the Coast Guard have made studies of this subject in connection with the need for protection of their stations, which contain information of value. In addition, the employees of those Services stationed at various points along the coast will be found to be particularly

well informed regarding weather, currents, and waves, and their effect on the shore line in their immediate vicinity.

There is still another important element in the assistance which the Federal Government can render. There are engineers in these various units of the Federal Service, whose normal work is closely related to this problem. The knowledge and experience they have thus acquired, if applied to studies of the kind here suggested, would aid materially in their solution.

The writer does not pretend to speak with authority for all Services. His experience has been, however, that they are uniformly zealous to serve the public in every possible way, and that they would gladly assist in this matter to the limit of their facilities. For example, to mention a single specific instance, the State of New Jersey, aroused by the alarming encroachment of the sea on its beaches, has instructed its Board of Commerce and Navigation to study the problem and undertake protective measures. That Board has organized an Advisory Board of Engineers to co-operate with it, which Board includes representatives of two Federal Services, the Army Engineers and the Coast and Geodetic Survey.

What the Government is doing for New Jersey, it will be equally willing to do for other States.

A. H. Rhett,* Assoc. M. Am. Soc. C. E. (by letter).—The fact is now established that concrete in contact with sea water undergoes disintegration under certain conditions, and the test conducted by the Aberthaw Construction Company, a final report on which has been made by Mr. Wason, has done much to establish this fact. Opinions seem to differ as to the cause of this disintegration, but the writer believes that the results of the various tests and examinations indicate the cause quite definitely.

It has been stated† that:

"Hydraulic cement is readily decomposed if intimately exposed to the chemical action of various sulphate and chloride solutions as contained in saline and alkaline waters, if the solutions are of sufficient strength, * * *. The cause of this disintegration is not certain, though it is almost universally believed that it is the reaction of the sulphate of magnesium of the sea water with the lime of the cement (formed during setting)."

It is also stated in that paper that: "The most soluble element of the cement is lime," and that the action is more vigorous in set cements than in unset cements, because "in set cements a considerable part of the lime has been already hydrated." And, further, that "contrary to the opinion of the many, there is no apparent relation between the chemical composition of a cement and the rapidity with which it re-acts with sea water."

The Aberthaw test confirms the statements just mentioned in demonstrating that the specimen to which hydrated lime had been added, showed very active disintegration. These findings have been confirmed by other investigators and seem to establish the fact that the disintegration of concrete by sea water is caused by the reaction of the sulphate and chloride solutions

* Gen. Sales Mgr. and Engr., Toch Brothers, New York City.

† "Action of the Salts in Alkali Water and Sea Water on Cements," U. S. Bureau of Standards, *Technologic Paper No. 12.*

of the water with the free lime (formed during setting) present in the concrete. Investigation has further disclosed that disintegration occurs principally where the concrete is alternately wet and dry, as between the high and low-water marks. In some cases, however, even under these conditions, disintegration does not occur. The question, therefore, is: If concrete contains lime, which is disastrously affected by ingredients in sea water, why does disintegration occur chiefly where the concrete is alternately wet and dry, and, even then, does not always occur? The answer can be summed up in one word—porosity.

It would seem that sea water is not sufficiently strong in the sulphate and chloride solutions to start this chemical action. If, however, the concrete is porous, each flushing fills the pores, and each drying out leaves a minute deposit of the salts. This process, repeated indefinitely, eventually produces a sufficiently concentrated solution in the pores to produce the chemical reaction and accompanying crystallization which disrupts the concrete. This action has been found to penetrate deep into the concrete—as much as 4 or 5 ft. It should be clear, then, that if concrete can be made non-porous, it will not be subject to this action of concentrated solutions and, therefore, will be stable in sea water.

L. F. BELLINGER,* M. AM. SOC. C. E. (by letter).—The U. S. Bureau of Standards has investigated the action of sea water on concrete structures, and the report of Messrs. Wig and Ferguson on this subject has been published by the Bureau of Yards and Docks of the Navy Department,† in which reference is made to the various concrete quay walls in the Brooklyn Navy Yard, that have been built at various times during the last thirty years. Pertinent extracts from the report are, as follows:

"The condition of the structures built between 1901 and 1904 is in striking contrast with those built prior to this period. * * * None of these walls has been repaired and, with one exception, the wall at berth 17-*h*, none is appreciably deteriorated."

* * * * * * * *

"Much of the repair work and replacement work which was carried on subsequent to 1912 has deteriorated to a considerable extent. In some cases, as at wall berth 1-*a*, rebuilt in 1913, the repair work is in need of extensive repairs now."

* * * * * * * *

"There are defects 'in the case of the reinforced concrete in Piers *E* and *F*, in which the cracking of the concrete is due to the corrosion of the embedded reinforcement which is uncoated and not embedded to a sufficient depth.'"

The report shows that the structures built during the period, 1901-04, are better than any built before or since, in that Navy Yard. After fifteen years, no repairs were necessary to such structures, whereas, after three years, extensive repairs were needed in a quay wall that was rebuilt in 1913. It appears desirable, therefore, to examine in detail the conditions that provided a concrete quay wall on which no repairs were needed after thirteen years, par-

* Lt.-Commander, C. E. C., U. S. N.; Public Works Officer, U. S. Naval Submarine Base, New London, Conn.

† *Bulletin No. 28*, October, 1917.

ticularly as to the plasticity or workability of the concrete mixture and the workmanship required in placing it.

The materials were ordinary, high-grade aggregate and cement, secured about New York City on competitive bids. In his paper, Mr. Wason shows well the effect of too little water in concrete for use in sea water. Numerous laboratory experiments at various universities show the great deteriorating effects of too much water in concrete in any location. Both too much and too little water are dangerous, because when concrete is too dry or too wet it is more easily mixed than when it is "just right". A description of what is "just right", as determined by long experience, is the purpose of this discussion.

In their report, previously mentioned, Messrs. Wig and Ferguson describe the sea-wall concrete as being "plastic". That term is very good, but, in the West, the equivalent term is "ankle-deep concrete", and, in the East, the term, "quaking mixture", is used. Neither term is quite correct for describing physically that quality of the concrete placed in the Brooklyn Navy Yard in 1901-03. This concrete was a 1:2:4 mix that was turned three times on the mixing-board, shoveled into wheelbarrows, dumped into the forms, and rammed into place. A light ramming produced a jelly-like mass, into which the workmen's feet would sink from 1 in. to 2 in. The mortar facing was a 1:2 composition, placed by shovel next to the forms as concrete was placed directly back of it, thus securing a facing 2 in. thick integral with the concrete, but no precision was attempted on any plane of demarcation between the facing and the concrete. The degree of plasticity of the mortar was the same as that of the concrete. From the foregoing, it may be seen that less ramming was used than would have been necessary with a "quaking" mixture and, therefore, a little more water was used; also, that appreciably less water was used than in the Western "ankle-deep" concrete. Frequently, the tide rose over the soft concrete within 15 to 30 min. after it had been placed in the forms.

In 1895, the writer began the mixing of "concrete medium" with the disapproval and scorn of contractors and foremen. The explanation given then was that a certain quantity of water was necessary for the concrete properly to "set"; that less than that quantity would result in a weaker, more porous concrete; and that a slight excess of water would, by heavy ramming which was then the practice, force surplus water to the surface where it would either run off or evaporate. Although it was then considered as an unsound practice, it was insisted to be far better to have too much water than too little.

The first opportunity for mixing concrete "exactly right", occurred at the Brooklyn Navy Yard, in March, 1901. This "sticky" concrete was more difficult to handle than the dryer mixture and close inspection was necessary to secure concrete approximating closely the "exactly right" plasticity, because extra labor was required and this causes friction. Nevertheless, results were obtained that proved the great economy in repairs which is mentioned in the report of the Bureau of Standards.

The writer concurs in the conclusions of Mr. Wason regarding concrete for use in sea water. When the moisture freezes and swells, a grain of sand

or a pebble is forced out of place. Deterioration proceeds slowly in a mortar facing 2 in. thick as compared with the rate when the concrete itself is exposed to the freezing and thawing process.

The Standard Cement Specifications of the Navy Department have provided ample protection against chemical action. Abrasion from floating ice and from wave action is certain to occur. It will progress more rapidly in the concrete than in the mortar facing, because larger particles will be removed. Mortar facing from low tide to high tide should be thicker than in other parts of the wall, and some facing should extend to the top of the wall and across the coping, unless a stone coping is used.

The record of these particular quay walls and of recent laboratory tests have proved that the quantity of water used in mixing is of the greatest importance. The kind of water is of little importance; in some of the walls, fresh water was used for mixing, but in most of them East River water was used.

Failures in some seaside concrete structures and in the Boston Navy Yard samples show well what not to do. Sloppy concrete and dry concrete are thereby proved to be taboo. This discussion has been written with the desire to assist in showing what to do to secure the most durable concrete in sea water.

D. E. Hughes,* M. Am. Soc. C. E. (by letter).—Eight large series of tests of 1:3 sand briquettes were made years ago, on account of the San Pedro breakwater. All these tests were similar, except in kind of water used in mixing and in time of immersion. Series *A*, *B*, *C*, and *D* were mixed with sea water. After 24 hours, Series *A* and *B* were immersed, *A* in sea water and *B* in fresh water. Immediately on being made, Series *C* and *D* were immersed, one in sea water and the other in fresh water. Series *E*, *F*, *G*, and *H* were mixed with fresh water, *E* and *F* being immersed after 24 hours in fresh water and sea water, respectively, and *G* and *H* being placed immediately, *G* in fresh water and *H* in sea water. In all cases, the briquettes remained in their moulds 24 hours without being removed from the glass plates on which they were made.

All these briquettes developed practically equal strengths, except those of Series *H*, which, at every age and without exception, were decidedly weaker. The opinion was reached that, in Series *H*, mechanical interference with setting resulted from the sea water displacing the lighter fresh water, and this led to the advice to use sea water in the mixing of concrete that is to be exposed to sea water before setting. In large masses of mortar or concrete, the displacing of fresh water by salt water would be less complete and the weakening less pronounced.

Respecting consistency of concrete, the depth one sinks into fresh concretes, even of equal wetness, depends on whether there is much, little, or no excess of mortar, and whether the aggregate is of broken stone or the more easily displaced rounded gravel. Therefore, such expressions as "ankle-deep", etc., are hardly indicative of relative wetness, except in the case of concretes that otherwise are equal.

* Civ. Engr., San Pedro, Calif.

As to rusting of reinforcement, it can be stated that iron and steel long embedded in clay or mud under the sea have been found to be well preserved, because the first water to reach them had only a limited effect, and the clay prevented displacements by succeeding quantities to repeat the attack. Probably, the comparative freedom of reinforced concrete from cracking below low tide is similarly explained. At higher elevations, both gravity and evaporation operate to make room for fresh water to continue the rusting and swelling of steel.

In Southern California, the earliest failures are above high tide where concrete is reached only by salt air and spray and where no animal or vegetable matter or other scum attaches to seal the pores against the renewal of the waters.

The need of guarding against rock-borers may also be mentioned. In the office of J. W. Ludlow, M. Am. Soc. C. E., Harbor Engineer, Port of Los Angeles, there are specimens of concrete attacked by such borers. These specimens were obtained from concrete pile jackets placed 14 years ago and recently removed on account of channel widening at East San Pedro. These borers make small entry holes, but they immediately enlarge to 1 in. in diameter, and some of them have extended through the 3-in. jackets.

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Transactions, Vol. LXXXVI

Page 477, Article 11, line 11, for "net", read "the".

Page 478, Article 108, substitute the following:

"*108.—Lateral Forces.*—The lateral forces shall be assumed as moving loads of 30 lb. per sq. ft. on one and one-half times the vertical projection of the structure plus 10% of the specified train load on one track, or for 50 lb. per sq. ft. on the same surface for the unloaded structure, whichever gives the larger stress".

Page 478, Article 112, omit last sentence beginning "The connections of such members, * * * ".

Page 480, Article 304, add paragraph, as follows:

"The struts at the base of viaduct towers shall be of sufficient strength to slide the movable shoes when the track is unloaded".

Page 486, Section 6, add Articles 605 and 606, as follows:

"*605.—Marking and Shipping.*—Pins and small parts shall be protected for shipment. Rivets and bolts of one length and diameter, also loose nuts or washers of each size, shall be packed separately. The contents of each package shall be marked thereon.

"*606.*—Members weighing more than 5 tons shall have the weight marked thereon."

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

Paper No. 1518

SPECIAL COMMITTEE ON SPECIFICATIONS FOR BRIDGE DESIGN AND CONSTRUCTION

FINAL REPORT ON SPECIFICATIONS FOR DESIGN AND CONSTRUCTION OF STEEL RAILWAY BRIDGE SUPERSTRUCTURE*

COMMITTEE

HENRY B. SEAMAN, *Chairman*,

J. H. AMES,
V. H. COCHRANE,
J. E. GREINER,
O. E. HOVEY,
C. W. HUDSON,
E. F. KELLEY,
M. S. KETCHUM,
S. B. SLACK,
I. F. STERN,
F. E. TURNEAURE,

C. R. HARDING, *Secretary*.

November 18, 1922.

* Presented to the Annual Meeting, January 17, 1923.

To the President and Board of Direction,

American Society of Civil Engineers:

The Special Committee on Specifications for Bridge Design and Construction has held three meetings since its report* of November 16th, 1921, was presented to the Annual Meeting of the Society on January 18th, 1922, and has completed its Specifications for Steel Railway Bridge Superstructure.

The Tentative Specifications for Steel Railway Bridges which were submitted with the report mentioned, have been fully discussed,† and the specifications presented herewith are the result of the Committee's interpretation of such discussions.

It seemed advisable that these specifications should not follow the exact lines of specifications previously offered by other technical societies, as the purpose of this Society is advisory rather than administrative. For that reason, the subject is treated in a general manner, instead of entering into explicit details which should be left to the individual engineer in the execution of work under his direction. The Committee has reviewed the subject again from its origin, has considered all suggestions offered, and, in formulating these final specifications, has endeavored to cover the fundamental principles of design, leaving to each engineer the privilege of adding such details as may particularly meet the condition of his practice or the requirements of his railroad.

Although the specifications follow what, for many years, has been considered good practice, the natural progress of more recent experience and study has also been recognized. They represent, it is believed, the most advanced practice that is justified at the present time.

It is considered desirable that a specification emanating from this Society should apply to spans of indefinite length, that such details as the number of trusses to be used, and other matters of varied opinion, should be left to individual judgment, as influenced by local or special conditions, and even in a matter such as the clearance diagram, it is suggestive rather than final.

For estimating the dead load, the weights of gravel and ballast have been classed together as 120 lb. per cu. ft., and all timber has been specified as 5 lb. per ft. B. M., as treated timber may be introduced at any time during the life of the bridge. The weight of steel rails, with fastenings, has been specified as 200 lb. per lin. ft. of track.

Perhaps the greatest advance has been made in the provision for live load. The Class M engine, proposed by D. B. Steinman, M. Am. Soc. C. E.,‡ has so many advantages for heavy loading that the way has been opened for its gradual introduction, if desired. Suggestions that similar results might be obtained by using heavy Class E engines, with relatively lighter train loads, do not conform to the present tendency toward heavier cars. The typical engines, E-10 and M-10, have been used as a basis, and moment diagrams have been inserted in the Appendix, as from these, any proportional increase may be readily produced.

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* *Proceedings*, Am. Soc. C. E., December, 1921, Papers and Discussions, p. 683.

† For abstracts of these discussions, see p. 532.

‡ See p. 606.

Much attention has been given by the Committee to the subject of impact. Various formulas were studied and compared with the tests, but none was found to conform with equal satisfaction to the tests of chord and of web members. It was thought that the information may not yet be sufficiently complete to justify a change from the formula which has been in service many years and has produced serviceable bridges.

It has been well recognized by those who have the responsibility of the maintenance of old bridges that they should be properly proportioned for the ultimate load, as an overloaded bridge will not receive equal service by any other method; some members of the truss might be relatively weak and require the condemnation of the entire bridge. As a step toward meeting this condition, Article 106, on overload, has been inserted. There is great reluctance in adopting the overload stress of 24 000 lb. per sq. in., but it is believed that when the stress of 20 000 lb. per sq. in. is exceeded, a "slow order" will be placed on the bridge, and the structure watched for signs of deterioration. It is recognized, however, that the bridge should be proportioned for extreme contingencies.

In the matter of longitudinal force, attention is called to the error of past practice, which has been honored more in its breach than in its observance. The coefficient of friction, estimated at 20%, has been the basis of all specifications for the sudden starting or stopping of trains; but the fact that the brakes on freight cars are designed so that the wheels will not skid under an empty car, limits their efficiency to 20% of that weight, and as a loaded car may weigh four times as much as an empty one, the braking power, being 20% of the empty car, will be only 5% of the loaded car for which the bridge is designed. The specification of 10% for sliding friction of the train is, therefore, conservative.

After a study of the full-sized column tests and the derivation of various column formulas, keeping in mind the requirements of long-span bridges and the rating of existing bridges, the Committee adopted the Euler-Rankine column formula, $p = \dfrac{16\,000}{1 + \dfrac{l^2}{13\,500\,r^2}}$, for design, believing it to be the most practicable formula of scientific origin, long established in practice, and best applicable to unusual or extraneous conditions. The use of the right-line formula, $p = 16\,000 - 70\,\dfrac{l}{r}$, would be permitted under these specifications, but if this right-line formula had been specified, it would have precluded the use of the Euler-Rankine formula with its many advantages.

Although the derivation of the Euler-Rankine formula makes it applicable to columns of indefinite length, it has been thought that, for vibrating structures, the length of the column should be limited so as to insure rigidity under sudden motion. The assumed limits placed in Article 202 are for this purpose, and may be modified later, if experience or judgment so dictates.

In providing for the increased allowable stress on steel of greater strength than that of the structural grade, it is recognized that the modulus of elasticity

of the stronger steel is practically unchanged, and the Euler-Rankine formula, with a modified numerator and denominator, as specified, is equally applicable. For the stronger steel, in which the yield point may approach its ultimate strength, the proportional increase of the allowable stress may be so close to the ultimate as to encroach on a proper factor of safety. For this reason, the increase has been restricted in application to a yield point of not more than 70% of the ultimate strength.

In his discussion* of the strength of lattice-bars, Otis E. Hovey, M. Am. Soc. C. E., has assumed that the curve of flexure of the column was a parabola and determined the numerator of his formula to be $4\ P\,l$. If the curve had been assumed to be that of a sinusoid, the numerator would have been $\pi\ P\,l$. It may be assumed, therefore, that an intermediate value of $3.357\ P\,l$ may be used for the numerator and thus simplify the formula to $R = \frac{P\,l}{4\,000\,y}$. This formula has been adopted by the Committee.

The spacing of stiffeners on plate girders does not justify undue refinement, and it is believed that the simple clause adopted will meet every purpose.

The specification for workmanship has been condensed as much as seemed to be practicable to meet the requirements of first-class work. The requirement for sub-punching and reaming has been generally avoided in order to save unnecessary expense, but it has been found that even the accuracy of the multiple punch will not produce a large number of plates that may be satisfactorily assembled without reaming. When the plates exceed five in number, it is deemed advisable to sub-punch and ream.

Heretofore, the permissible difference between pin-hole and pin, 5 in. in diameter or less, has been restricted to $\frac{1}{50}$ in. This restriction has been extended to apply to pins 8 in. in diameter, or less.

The Committee presents the Specification for Steel Railway Bridge Superstructure in completed form, and recommends its adoption.

Respectfully submitted,

For the Committee,

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* See p. 575.

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SPECIFICATIONS FOR STEEL RAILWAY BRIDGE SUPERSTRUCTURE

SECTION A.—GENERAL

1.—These specifications cover the design of fixed span bridges of all lengths and require first-class design, material, and workmanship.

2.—Material.—Bridges shall be made wholly of structural steel, except where otherwise specified. Castings shall be of steel, unless cast iron is specifically authorized by the Engineer.

3.—Notice to Engineer.—The Contractor shall give ample notice to the Engineer of the beginning of rolling and of shop work, so that inspectors may be provided. No material shall be rolled, or work done, before the Engineer has been notified where the orders have been placed.

4.—Type of Bridge.—The type of bridge to be used for various span lengths may be as follows:

Rolled beams up to	30 ft.
Plate girders from	30 to 125 ft.
Riveted trusses from	100 ft. and more
Pin-connected trusses from	150 ft. and more

5.—Spacing of Girders and Trusses.—The girders of deck bridges and stringers, where two are used on each track to carry open floors, shall not be spaced closer than 6 ft. 6 in., center to center.

The width between centers of trusses or girders shall be sufficient to give lateral stiffness and to prevent overturning by the specified lateral forces, and, in no case, shall be less than one-twentieth of the span.

6.—Depth Ratios.—The depth of trusses preferably shall be not less than one-tenth of the span. The depth of plate girders preferably shall be not less than one-twelfth of the span. The depth of rolled beams used as girders and the depth of solid floors preferably shall be not less than one-fifteenth of the span. If less depths are used, the section must be increased so that the maximum deflection will not be greater than if these limiting ratios had not been exceeded.

7.—Clearance.—The clearance on straight track shall not be less than that shown in Fig. 1. On curves, additional provision shall be made for a car, 80 ft. long, 60 ft. between truck centers, 14 ft. high above the top of a 6-in. rail, and with allowance for super-elevation of the outer rail. Unless otherwise specified, the super-elevation of the outer rail shall be $\frac{3}{4}$ in. for each degree of curvature, with a maximum of 6 in.

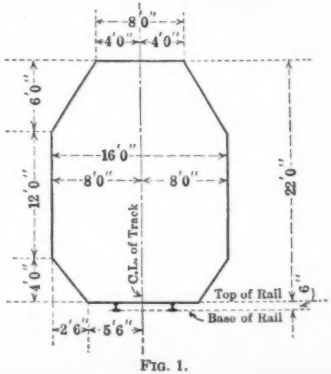


FIG. 1.

8.—Skew Bridges.—In skew bridges without ballasted floors, the end stringers or end girders for each track shall be square to the line of track.

9.—Timber Floor.—Ties shall not be less than 10 ft. long, spaced about 6 in. apart, and secured against bunching. If subjected to bending, the stress in the extreme fibers of ties shall not exceed the unit stress for timber given in Article 203, assuming a maximum wheel load, with 100% impact, to be distributed over three ties.

10.—Ballasted Floors.—Ballasted floors shall have at least 6 in. of ballast under the ties, and the ballast shall

be assumed as level with the base of rail, the weight of the ties being neglected. The live load on each tie shall be assumed as one-third of the maximum axle load distributed over a width of 10 ft.

11.—Dimensions for Calculations.—For the calculation of stresses, the length shall be:

For trusses and girders, the distance between centers of bearing;
For floor-beams, the distance between centers of trusses;
For stringers, the distance between centers of floor-beams;

and the depth shall be:

For pin-connected trusses, the distance between centers of pins;
For riveted trusses, the distance between centers of gravity of chords;
For plate girders, the distance between centers of gravity of flanges (not to exceed out to out of flange angles) unless net section modulus is used.

12.—Spans with floor systems shall preferably have end floor-beams.

Section 1.—Loads and Stresses

101.—Loads.—Stresses shall be shown separately for the following: Dead load, live load, impact, centrifugal force, and lateral and longitudinal forces. Members shall be proportioned for the combination giving maximum sections, except as otherwise provided.

102.—Dead Load.—In estimating the weight of the structure, for the purpose of computing dead load stresses therein, the following unit weights shall be used:

Steel	490	lb. per cu. ft.
Concrete	150	" " " "
Sand, gravel, and ballast	120	" " " "
Asphalt-mastic and bituminous macadam	150	" " " "
Granite	170	" " " "
Paving bricks	150	" " " "
Timber	5	" per ft. B. M.

The rails, inside guard rails, and fastenings shall be assumed to weigh 200 lb. per lin. ft. for each track.

103.—Live Loads.—The live load for each track shall consist of typical engines followed by a uniform train load, according to either Class E series, or Class M series, as may be specified by the Engineer. It shall be a multiple of one or the other of the loads with wheel spacings, as shown in Figs. 2 and 3.

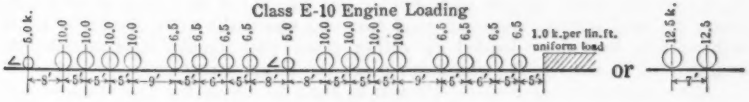


FIG. 2.

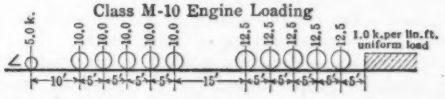


FIG. 3.

Loading E-60 or Loading M-50 is recommended for main-line bridges of American railways.

104.—Impact.—The dynamic increment of the live load shall be added to the maximum computed live load stresses and shall be determined by the formula:

$$I = S \frac{2\,000 - L}{1\,600 + 10\,L},$$

in which,

I = impact or dynamic increment to be added to live load stresses;
S = computed maximum live load stress;
L = loaded length of track, in feet, producing the maximum stress in the member. For bridges carrying more than one track, the aggregate length of all tracks producing the stresses shall be used.

Impact shall not be added to stresses produced by longitudinal and lateral or wind forces.

105.—For bridges designed exclusively for electric traction, impact shall be taken as one-third of that given by the impact formula.

106.—Allowance for Increase of Live Load.—Wherever an additional counter would be required, or reversal of stress would be caused, or heavier dimensioning would result, a member and its details shall be designed for a live load 50% greater than that given in Article 103, with an allowance of 50% increase in unit stresses.

107.—Centrifugal Force.—Structures on curves shall be designed to resist the centrifugal force of the live load applied 7 ft. above base of rail, as computed by the following formula:

$$C = \frac{0.067\ W\ V^2}{R}$$

in which,

C = horizontal centrifugal force;
W = live load, including impact;
R = radius of curve (5 730 ÷ degree of curve);
V = speed, in miles per hour = (60 — 2½ times the degree of curve).

108.—Lateral Forces.—All spans shall be designed for a lateral force on the loaded chord of 200 lb. per lin. ft. plus 10% of the specified train load on one track, and 200 lb. per lin. ft. on the unloaded chord, these forces being considered as moving.

109.—Wind Force.—Viaduct towers shall be designed for the one of the following loads that causes the greater stress:

Viaduct towers shall be designed for a force of 50 lb. per sq. ft. on one and one-half times the vertical projection of the structure unloaded; or 30 lb. per sq. ft. on the same surface plus 400 lb. per lin. ft. of structure applied 7 ft. above the rail for the assumed wind force on the train when the structure is either fully loaded or loaded on either track with empty cars assumed to weigh 1 200 lb. per lin. ft., whichever gives the larger stress.

110.—Longitudinal Force.—Provision shall be made for the starting and stopping of trains, with a coefficient of friction on the engine drivers at 20% and on the remainder of the train at 10 per cent.

111.—Temperature.—On simple spans, provision shall be made for expansion due to a variation in temperature of 120° Fahr.

112.—Alternate Stresses.—Members subject to alternate stresses of tension and compression shall be proportioned for the kind of stress requiring the larger section. If the alternate stresses occur in succession during the passage of one train, as in stiff counters, each stress shall be increased by 50% of the smaller. The connections of such members shall be proportioned in all cases for the sum of the stresses thus increased.

If the live load and dead load stresses are opposite in character, only two-thirds of the dead load stress shall be considered as effective in counteracting the live load stress. This reduction of dead load shall not be made in proportioning members subject to alternate stresses.

113.—Combined Stresses.—Members subject to both axial and bending stresses shall be proportioned so that the combined fiber stresses will not exceed the unit stresses given in Article 201. In members continuous over panel points, three-fourths of the bending stress computed as for simple beams shall be added to the axial stress.

114.—For stresses produced by combination of longitudinal or lateral and wind forces with live load, dead load, impact, and centrifugal force, the unit stresses may be increased 25% over those specified in Article 201. When secondary stresses are included, the unit stress may be increased 33⅓ per cent. In no case shall the section be less than that required for dead load, live load, impact, and centrifugal force at the unit stresses specified in Article 201, or less than that required if secondary stresses are not considered.

115.—Secondary Stresses.—Secondary stresses shall be avoided where possible in designing and detailing. In ordinary trusses without sub-paneling, secondary stresses due to distortion need not be considered in any member the width of which measured parallel with the plane of flexure is less than one-tenth of its length. All other secondary stresses shall be considered.

Section 2.—Unit Stresses

201.—The unit stresses to be used in proportioning the several parts of the structure shall be as follows:

Allowable Stresses for Structural and Rivet Steel:

	Kips per* square inch.
Tension	16.0
Compression (one diameter)	16.0

Compression on columns:

$$p = \frac{16.0}{1 + \frac{l^2}{13\,500\,r^2}}$$

in which:

p = allowable unit stress;
l = length of member, in inches;
r = least radius of gyration of member, in inches;

but not to exceed the value for $\frac{l}{r} = 40$.

Bending in extreme fibers of rolled shapes, built sections, and girders, net section	16.0
Bending in extreme fibers of pins	24.0
Shear in plate-girder and I-beam webs, net sections	12.0
Shear in pins and power-driven rivets	12.0
Shear in turned bolts and hand-driven rivets	10.0
Bearing on pins, power-driven rivets, outstanding legs of stiffener angles, and other steel parts in contact	24.0
Bearing on turned bolts and hand-driven rivets	20.0
Bearing on countersunk rivets. Only one-half the countersink shall be computed as bearing surface.	
Bearing on rollers per linear inch	0.6 d

* 1 k. = 1 000 lb.

in which,

d = diameter of roller, in inches.

For cast-steel shoes and pedestals, the allowable unit stress for structural steel will apply.

For members composed of steel of greater strength than structural grade the allowable stresses may be increased in proportion to the higher yield point of the stronger steel, provided the yield point is not more than 70% of the ultimate strength. In the column formula, the fractional part of the denominator should be increased in the same ratio.

202.—Limiting Length of Members.—The lengths of main compression members shall not exceed 100 times their least radius of gyration, and those for wind and sway-bracing 120 times their least radius of gyration.

The lengths of riveted tension members shall not exceed 200 times their least radius of gyration.

203.—Allowable Fiber Stress on Wooden Cross-Ties.—(The maximum wheel load with 100% impact, will be distributed over three ties):

	Kips per square inch.
White oak and dense yellow pine	2.0
Dense Douglas fir	1.5
White pine, ordinary yellow pine, and spruce	1.2

204.—Allowable Pressure on Masonry.—

	Kips per square inch.
Granite masonry	0.8
Limestone and sandstone (good quality)	0.4
Concrete (1:2:4)	0.6

Section 3.—Details of Design

301.—Thickness of Material.—In bridges designed for live loads of more than E-40, material less than $\frac{3}{8}$ in. in thickness shall not be used, except for fillers.

In bridges designed for live loads of E-40 and less, material $\frac{5}{16}$ in. in thickness may be used.

302.—Open Details.—Details shall be arranged so as to give free access for inspection and painting. Water-pockets shall be avoided.

303.—Compression Members.—Compression members shall be designed so that the metal shall be concentrated as far as feasible in the webs and flanges, and so that the center of gravity of the section may be as near the center line of the member as practicable.

304.—Rigid Members.—Hip verticals and members performing similar functions, and the two end panels of the bottom chords of single-track pin-connected bridges preferably shall be rigid.

305.—Net Sections.—Net sections shall be used in all cases in calculating tension members; and, in deducting rivet holes, they shall be taken as $\frac{1}{8}$ in. larger than the nominal diameter of the rivet. The weakening effect of a staggered rivet shall be allowed for by deducting from the transverse section a strip, w in width, as given by the formula:

$$w = h - \frac{s^2}{4\,g},$$

in which,

w = width, in inches, of strip to be deducted;
h = diameter of rivet hole, in inches;
s = stagger, or longitudinal spacing of rivet with respect to rivet on last gauge line, in inches;
g = distance between gauge lines, or transverse spacing, in inches.

306.—Pin-connected riveted tension members shall have a net section back of the pin-hole, parallel to the axis of the member, of not less than the required net section of the body of the member, and shall have a net section through the pin-hole at least one-third larger than the required section. Riveted tension members shall be stitch-riveted where necessary to make a compact member.

307.—Maximum Length of Rivet.—The total thickness of plates connected by rivets shall not exceed six times the nominal diameter of the rivet to be used.

308.—Pitch of Rivets.—Rivets shall be proportioned by their nominal diameter. They shall not be spaced, center to center, closer than 3 diameters nor farther apart, in the direction of the stress, than sixteen times the thickness of the thinnest plate connected, nor farther apart, at right angles to the line of stress, than thirty times that thickness, except in the cover-plates of compression members, where the spacing may be forty times the thickness of the thinnest plate. The pitch of rivets at ends of built compression members shall not be more than 4 diameters, for a distance of one and one-fourth times the width of the member.

309.—Edge Distance of Rivets.—Rivets shall not be spaced closer to sheared edges than 1¾ diameters, nor to rolled or planed edges, except in flanges of beams and channels, than 1½ diameters, nor farther from the edge than eight times the thickness of the plate.

310.—Built Chord Splices.—Built chords subjected to compression only, when faced for bearings, shall be spliced on four sides sufficient to hold the abutting members accurately in place and to transmit at least 25% of the stress through splice-plates. Generally, they shall be spliced as near to panel points as practicable. All other joints shall be fully spliced.

311.—Extra Rivets in Packing.—Filler-plates through which stress is transmitted, or splice-plates not in direct contact with the parts they connect, shall be extended for extra riveting.

312.—Latticing Compression Members.—All segments of members in compression, connected by latticing only, shall have batten-plates at each end. The thickness of such plates shall not be less than one-fortieth of the distance between the rivets connecting them to the compression member. In no case shall the length of such batten-plates be less than 1¼ times the width of the member. Where intermediate batten-plates are used, their length shall be at least three-quarters of the width of the member.

The distance between the connections of latticing shall be such that the individual members between them, composing the column shall be relatively stronger than the column as a whole.

313.—Tension members composed of shapes shall have their separate segments connected by batten-plates, or by batten-plates and lattice-bars.

314.—Lattice-Bars.—Single lattice-bars shall generally be inclined at an angle of 60° with the axis of the member, and double lattice-bars at an angle of 45°, riveted at the intersection. Single lattice-bars shall have a thickness of not less than one-fortieth, and double lattice-bars not less than one-sixtieth, of the distance between rivets connecting them to the compression member.

315.—The latticing of compression members shall be proportioned to resist shearing stress normal to the member not less than that calculated by the formula:

$$R = \frac{P\,l}{4\,000\,y}$$

in which,

R = normal shearing stress, in pounds;
P = strength of column as a compression member, expressed in pounds;
l = length of column, in inches;
y = distance from neutral axis to extreme fiber, in inches.

In a compression member with a cover-plate, the cover-plate will be assumed to take one-half the shear.

316.—The diameter of the rivets shall not exceed one-third the width of the bar.

317.—Reinforcing Pin-Holes.—Where necessary, pin-holes shall be reinforced by plates. These plates shall be connected so as to distribute the bearing pressure proportionately over the full cross-section with a minimum of eccentricity. At least one full-width plate on each segment shall extend 6 in. beyond the near edge of the batten-plate.

318.—Forked Ends.—Forked ends of compression members should be avoided. Where forked ends are used, a sufficient number of pin-plates shall be provided to give each jaw the full strength of the compression member. At least one of these plates shall extend to the far edge of the batten-plates, and the others not less than 6 in. beyond the near edge of the batten-plates.

319.—Floor-Beam Connections.—Floor-beams preferably shall be square to the girders or trusses, and shall be riveted directly to the girders or to the posts of the trusses.

320.—Lateral Bracing.—Through bridges shall be provided with post brackets at the intermediate panel points, of sufficient strength to maintain the panel in a vertical position under the specified wind pressure. When the height of the top chord is more than 26 ft. above the floor, an overhead system of sway-bracing shall be used.

321.—In either deck or through bridges, the end sway-bracing shall be proportioned to carry the entire upper lateral stress to the support, through the end posts of the truss.

Rigid lateral bracing is preferred and no connection shall have less than three rivets.

322.—Viaduct towers shall be braced both transversely and longitudinally, with a double system of rigid diagonals with riveted connections. Longitudinal and transverse struts shall be placed at caps and bases and at all intermediate panel points. All bracing connections shall be made with gusset-plates. The struts at the base of towers shall be strong enough to slide the movable shoes and provide expansion, with the structure unloaded.

323.—Plate Girders.—Plate girders shall be proportioned by assuming that the flanges are concentrated at their centers of gravity, but in no case beyond the back of the flange angles. One-eighth of the gross section of the web may be considered as flange area, provided the web is properly spliced to transmit its bending moment. For unusual sections, the net section modulus shall be used.

324.—Compression Flanges.—The gross section of compression flanges of plate girders or I-beams shall not be less than the gross section of the tension flanges, but the stress per square inch shall not exceed:

$$16.0\ K - 0.15\ \frac{l}{b},$$

in which,

l = length of unsupported flange, between lateral connections or knee-braces, in inches; and

b = width of flange, in inches.

325.—When flange plates are used, at least one plate on each flange shall extend the full length of the girder, and on through bridges, an end and corner cover-plate shall be used. Any additional flange plates shall be of such length as to allow two rows of rivets of the regular pitch to be placed at each end of the plate, beyond the theoretical point required, and there shall be a sufficient number of rivets at the ends of each plate to transmit its stress value before the theoretical point of the next outside plate is reached.

326.—Flange plates which are spliced, shall be properly covered by extra material equal in section to the material spliced. Flange angles shall have angle splices. There shall be a sufficient number of rivets on each side of the splice to transmit the stress value of the parts cut.

327.—The webs of plate girders, wherever cut, shall be fully spliced for shear and bending.

328.—Stiffeners.—Stiffener angles shall be placed at end bearings and at points of concentrated load. Such stiffeners shall not be crimped and shall have outstanding legs proportioned for bearing and extending as nearly as practicable to the edge of the flange angles.

329.—Intermediate Stiffeners.—Webs shall be stiffened by angles riveted thereto in pairs, with outstanding legs not exceeding sixteen times their thickness and not less than 2 in. plus one-thirtieth of the depth of the girder. Intermediate stiffeners shall be placed at intervals not exceeding the depth of the web, nor 6 ft.

330.—If the depth of the web between the flange angles or side-plates is less than fifty times the thickness of the web, intermediate stiffeners may be omitted.

331.—Expansion Rollers.—All bridges, 80 ft. or more in length, that bear on masonry, shall be provided with pin-bearing bolsters, and, at one end, shall have turned rollers not less than 4 in. in diameter, placed between two planed surfaces. Spans of less than 80 ft. shall be free to move at one end on planed surfaces without rollers.

332.—The nests of rollers shall be designed so as to prevent displacement.

333.—Bearings Anchored.—Trusses shall be secured against side motion on bearing plates and rollers. The bolsters shall be joined to the trusses and the bearing plates secured to the underlying supports by bolts. Anchor-bolts shall not be less than 1¼ in. in diameter, and shall extend not less than 18 in. into the masonry. Washers shall be used under the nut. Anchor-bolts subjected to tension, as in viaduct towers, shall engage 50% more masonry than is sufficient to counteract the uplift.

334.—Eye-Bar Packing.—Eye-bars shall not be packed out of line with the axis of the member more than $\frac{1}{32}$ in. to 1 ft. Members packed on pins shall be held against side movement.

335.—Camber.—The length of truss members shall be such that the camber will be equal to the deflection produced by the dead load plus the full train load, without impact.

Section 4.—Workmanship

401.—General.—All workmanship shall be equal to the best practice in modern bridge shops. The methods used and the character of the work done shall be such that the strength of the various members, as designed, shall be fully realized.

402.—Straightening.—When necessary, the material shall be straightened by methods that will avoid any excessive local internal stresses. Sharp kinks and bends may be cause for rejection.

403.—Shearing Structural Steel.—All shearing shall be done neatly and material exceeding ¾ in. in thickness, shall have ¼ in. planed from the sheared edges.

404.—Shearing Alloy Steel.—All shearing shall be done neatly and material exceeding ½ in. in thickness shall have ¼ in. planed from the sheared edges.

405.—Rivet Holes.—Material forming parts of a member composed of more than five thicknesses of metal shall be punched, or drilled, with a tool $\frac{1}{16}$ in. smaller than the nominal size of the rivets and reamed to full size after the parts are assembled. When the holes are made after the parts are assembled, they shall be drilled full size.

406.—Rivet Holes in Structural Steel.—Material forming parts of a member composed of not more than five thicknesses of structural steel may be punched $\frac{1}{16}$ in. larger than the nominal size of the rivets, whenever the thickness of the metal is equal to, or less than, the nominal size of the rivets, plus $\frac{1}{8}$ in. All metal thicker than the nominal size of the rivets, plus $\frac{1}{8}$ in., shall be drilled from the solid.

407.—Rivet Holes in Alloy Steel.—Material, $\frac{1}{2}$ in., or less, in thickness, forming parts of a member composed of not more than five thicknesses of alloy steel may be punched $\frac{1}{16}$ in. larger than the nominal size of the rivets. Material thicker than $\frac{1}{2}$ in. shall be punched, or drilled, with a tool $\frac{1}{16}$ in. smaller than the nominal size of the rivets and reamed to full size after the parts are assembled. All metal thicker than the nominal size of the rivets, less $\frac{1}{4}$ in., shall be drilled from the solid.

408.—Drifting.—Drifting to enlarge unfair holes will not be allowed.

409.—Field Rivet Holes.—All holes for field connections, except those for bracing and similar parts, shall be reamed or drilled to a metal templet, or while the members are temporarily assembled. When the reaming or drilling is done with the pieces assembled, the parts shall be match-marked and a diagram of the marks made.

410.—Dies.—Punch dies shall not be more than $\frac{3}{32}$ in. larger than the punch.

411.—Web-Plates.—Web-plates of girders which have no cover-plates, shall be flush with the backs of the top flange angles or project above them not more than $\frac{1}{8}$ in.

Web-plates of girders that have cover-plates may be $\frac{1}{2}$ in. less in width than the distance back to back of flange angles. The ends of web-plates at splices shall not be more than $\frac{1}{4}$ in. apart.

412.—Splice-Plates and Fillers.—Web splices, plates, and fillers shall fit within $\frac{1}{8}$ in. of the flange angles.

413.—Web Stiffeners.—Web stiffeners at bearings and points of concentrated loads on the flanges, shall be faced and brought to bearing against the flange angles.

414.—Lattice-Bars.—The ends of the lattice-bars with single rivets shall be neatly rounded.

415.—Floor-Beams and Stringers.—The main sections of floor-beams and stringers shall be of exact length after riveting, and the end-connection angles shall be accurately set to length, and square.

416.—Turned Bolts.—Generally, the use of bolts instead of rivets shall not be permitted. Wherever bolts are used, the holes shall be reamed parallel and the bolts turned to a driving fit, and the threads shall be entirely outside the holes. A washer not less than $\frac{1}{4}$ in. thick shall be used under the nut and the thread burred.

417.—Abutting Joints.—Abutting joints shall be fitted close. The abutting surfaces of compression joints which depend on contact bearing shall be accurately faced.

418.—Eye-Bars.—Eye-bars shall be made by a process that will produce a uniform product. Welds will not be allowed. The form of the heads may be determined by the dies at the works where the eye-bars are made, if satisfactory to the Engineer, but the manufacturers shall guarantee the bars to break in the body when tested to rupture. The thickness of the head shall not be more than $\frac{1}{8}$ in. greater than that of the bar, except by mutual agreement in special cases.

419.—Pin Clearance.—The diameters of pin-holes shall not exceed those of the pins by more than $\frac{1}{50}$ in. for pins 8 in. or less in diameter, nor more than $\frac{1}{32}$ in. for larger pins.

420.—Pins and Rollers.—Pins more than 7 in. in diameter shall be forged and annealed. Pins 9 in. or more in diameter shall have a 2-in. longitudinal hole bored through the axis. All pins shall be turned.

421.—Steel Castings.—Steel castings shall be free from large and injurious blow-holes and shall be annealed.

422.—Bed-Plates.—Expansion bed-plates shall be planed true and smooth. The finishing cut of the planer tool shall be in the direction of expansion. Cast wall-plates shall be planed on top.

423.—Screw-Threads.—Screw-threads shall be U. S. Standard. The nuts shall fit tight.

424.—Welds.—Welds will be allowed only where the unit stress is not more than one-half the ordinary specified working stress.

425.—Pilot Nuts.—Two pilot and two driving nuts shall be furnished for each size of pin.

426.—Field Rivets.—Field rivets shall be furnished in excess of the nominal number required to the amount of 10% plus 10 rivets of each size and length.

427.—Shop Painting.—All surfaces that come in contact or that are closed shall receive one coat of approved paint before being assembled. All steel shall be scraped free of scale and shall receive one coat of approved paint before leaving the shops.

428.—All machined surfaces shall receive a coat of white lead and tallow at the shop.

429.—Facilities for Inspection.—The Inspector shall be allowed access to all necessary parts of the works.

430.—Inspector's Authority.—The Inspector shall have the power to reject materials and workmanship which do not fulfill the requirements of these specifications; but, in case of dispute, the Contractor may appeal to the Engineer, whose decision shall be final.

431.—Rejection.—The acceptance of any material or finished members by the Inspector, shall not be a bar to their subsequent rejection if they are found to be defective.

Rejected material and workmanship shall be replaced promptly or made good by the Contractor.

Section 5.—Full-Size Eye-Bar Tests

501.—Material Tests.—When the acceptance of eye-bars depends on the results of full-size tests, the quality of the material, as proved by the specimen tests, shall be determined by the manufacturer.

502.—Number of Full-Size Tests.—The number and size of the bars to be tested shall be specified by the Engineer before the mill order is placed.

503.—Selection of Eye-Bars.—The test bars shall be the same section as the bars that are to be used in the structure and of the same length, if within the capacity of the testing machine. Test bars representing bars too long for the testing machine shall be selected from the full length bar material after the heads on one end have been formed, and shall then be cut and the second head formed to the greatest length that can be tested.

504.—Requirements for Full-Size Tests.—The requirement for full-size eye-bar tests shall be as follows:

	Structural steel annealed eye-bars.
Yield point, in pounds per square inch, minimum.....	30.0 k
Ultimate strength, in pounds per square inch, minimum.	55.0 k
Elongation percentage in 18 ft., minimum.....	12

The elongation shall be measured in the body of the bar including the fracture.

505.—Re-Tests.—When a bar fails to meet the requirements, two additional bars of the same size and from the same mill heat may be tested. If two of three bars tested fail, the bars of that size and mill heat shall be rejected.

506.—Payment for Eye-Bars.—Eye-bars thus tested which meet the requirements of these specifications, shall be paid for by the purchaser at the same unit price as for the structure. Bars which fail to meet the requirements of the specifications, and all bars rejected as a result of the tests, shall be at the expense of the Contractor.

Section 6.—Weighing and Shipping

601.—Weight Paid For.—The payment for pound-price contracts shall be based on the weight of metal in the fabricated structure, including field rivets shipped. The weight of erection bolts, field paint, and all boxes, crates, or other containers used for packing, together with sills, struts, and rods used for supporting members during transportation, shall be excluded.

Weights paid for shall be shop-scale weights unless otherwise provided. If specified in the contract or permitted by the Engineer, computed weights, as hereinafter provided, may be made the basis of payment.

602.—Variation in Weight.—If the weight of any member is more than 2½% less than the computed weight, it may be cause for rejection. This applies to both pound-price and lump-sum contracts.

If the total scale weight of any structure exceeds the computed weight by more than 2%, the weight in excess of 2% above the computed weight shall not be paid for.

603.—Weighing of Members.—Finished work shall be weighed in the presence of the Inspector, if practical. The Contractor shall supply satisfactory scales and shall perform all work involved in handling and weighing the various parts.

604.—Computed Weight.—The weight of steel shall be assumed at 490 lb. per cu. ft. The weight of cast iron shall be assumed at 450 lb. per cu. ft.

The weights of rolled shapes, and of plates up to and including 36 in. in width, shall be computed on the basis of their nominal weights and dimensions, as shown on the approved shop drawings, deducting for copes, cuts, and open holes.

The weights of plates wider than 36 in. shall be computed on the basis of their dimensions, as shown on the approved shop drawings, deducting for cuts and open holes. To this shall be added one-half of the allowed percentages of overrun in weight given in Article 711.

The weight of heads of shop-driven rivets shall be included in the computed weight, assuming the weights to be as follows:

Diameter of rivet, in inches:	Weight for 100 heads in pounds:
½	4.5
⅝	8.5
¾	14.5
⅞	21.5
1	31.0

The weight of castings shall be computed from the dimensions shown on the approved shop drawings, with an addition of 10% for fillets and overrun.

To the total computed weight of metal may be added an allowance of 0.4 of 1% for shop paint.

Section 7.—Structural Steel for Bridges

These specifications conform to the Standard Specifications for Structural Steel for Bridges of the American Society for Testing Materials.

701.—Steel Castings.—The Standard Specifications for Steel Castings (Serial Designation, A27), adopted by the American Society for Testing Materials, shall govern the purchase of steel castings for bridges. Unless otherwise specified, Class B castings, medium grade, shall be used.

I.—Manufacture

702.—Process.—The steel shall be made by the open-hearth process.

II.—Chemical Properties and Tests

703.—Chemical Composition.—The steel shall conform to the following requirements as to chemical composition:

		Structural steel.	Rivet steel.
Phosphorus	Acid	not over 0.06%	not over 0.04 %
	Basic	" " 0.04%	" " 0.04 %
Sulphur		" " 0.05%	" " 0.045%

704.—Ladle Analyses.—An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of carbon, manganese, phosphorus, and sulphur. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Article 703.

705.—Check Analyses.—Analyses may be made by the purchaser from finished material representing each melt. The phosphorus and sulphur content thus determined shall not exceed that specified in Article 703 by more than 25 per cent.

III.—Physical Properties and Tests

706.—Tension Tests.—

(*a*).—The material shall conform to the requirements as to tensile properties given in Table 1.

TABLE 1.

Properties considered.	Structural steel.	Rivet steel.
Tensile strength, in pounds per square inch	55 000—65 000*	46 000—56 000
Yield point, minimum, per square inch	0.5 tensile strength	0.5 tensile strength
Elongation in 8 in., minimum percentage	$\frac{1\,500\,000}{\text{Tensile strength}}$†	$\frac{1\,500\,000}{\text{Tensile strength}}$
Elongation in 2 in., " "	22	.....

* See Article 706, Paragraph (*b*).

† See Article 707.

(*b*).—In order to meet the required minimum tensile strength of full-size annealed eye-bars, the purchaser may determine the tensile strength to be obtained in specimen tests; the range shall not exceed 14 000 lb. per sq. in., and the maximum shall not exceed 74 000 lb. per sq. in. The material shall conform to the requirements as to physical properties other than that of tensile strength, specified in Articles 706(*b*), 707(*b*), and 708(*b*).

(*c*).—The yield point shall be determined by the drop of the beam of the testing machine.

707.—Modifications in Elongation.—

(*a*).—For structural steel more than $\frac{3}{4}$ in. in thickness, a deduction from the percentage of elongation in the 8 in. specified in Article 706(*a*), of 0.25% shall be made for each increase of $\frac{1}{32}$ in. of the specified thickness above $\frac{3}{4}$ in., to a minimum of 18 per cent.

(*b*).—For structural steel less than $\frac{5}{8}$ in. in thickness, a deduction from the percentage of elongation in the 8 in. specified in Article 706(*a*), of 1.25% shall be made for each decrease of $\frac{1}{32}$ in. in thickness below $\frac{5}{8}$ in.

708.—Bend Tests.—

(*a*).—The test specimen for plates, shapes, and bars, except as specified in Paragraphs (*b*), (*c*), and (*d*), shall bend cold through 180° without cracking on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. or less in thickness, flat on itself; for material more than $\frac{3}{4}$ in. to and including $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material more than $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

(*b*).—The test specimen for eye-bar flats shall bend cold through 180° without cracking on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. or less in thickness, around a pin the diameter of which is equal to the thickness of the specimen; for material more than $\frac{3}{4}$ in. to and including $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen; and for material more than $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to three times the thickness of the specimen.

(*c*).—The 1 in. by $\frac{1}{2}$-in. test specimen for pins, rollers, and other bars, when prepared as specified in Article 709, shall bend cold through 180° around a pin 1 in. in diameter, without cracking on the outside of the bent portion.

(*d*).—The test specimen for rivet steel shall bend cold through 180° flat on itself, without cracking on the outside of the bent portion.

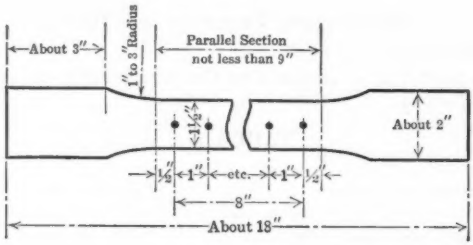


FIG. 4.

709.—Test Specimens.—

(*a*).—Test specimens shall be prepared for testing from the material in its rolled or forged condition, except when it is specified to be annealed, in which case the test specimens shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

(*b*).—Test specimens shall be taken longitudinally and, except as specified in Paragraphs (*d*), (*e*), and (*f*), shall be of the full thickness or diameter of the material as rolled.

(*c*).—Test specimens for plates, shapes, and flats may be machined to the form and dimensions shown in Fig. 4, or with both edges parallel; except that bend-test specimens for eye-bar flats may have three rolled sides.

(*d*).—Tension-test specimens for plates and eye-bar flats more than $1\frac{1}{2}$ in. in thickness, and bend-test specimens for plates more than $1\frac{1}{2}$ in. in thickness, may be machined to a thickness or diameter of at least $\frac{3}{4}$ in. for a length of at least 9 in.

(*e*).—Test specimens for bars more than $1\frac{1}{2}$ in. in thickness or diameter may be machined to a thickness or diameter of at least $\frac{3}{4}$ in. for a length of at least 9 in.; or tension-test specimens may conform to the dimensions shown in Fig. 5, in which case the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens may be 1 in. by $\frac{1}{2}$ in. in section.

(*f*).—Tension-test specimens for pins and rollers shall conform to the dimensions shown in Fig. 5. In this case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens shall be 1 in. by ½ in. in section.

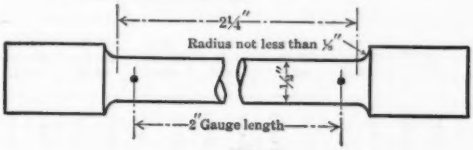


FIG. 5.

(*g*).—The tension-test specimen shown in Fig. 5 and the 1 in. by ½-in. bend-test specimen for pins and rollers shall be taken so that the axis is 1 in. from the surface; and for other bars more than 1½ in. in thickness or diameter, midway between the center and surface.*

(*h*).—The machined sides of rectangular bend-test specimens may have the corners rounded to a radius of not more than $\frac{1}{16}$ in.

(*i*).—Test specimens for rivet bars which have been cold drawn, shall be normalized before testing.

710.—Number of Tests.—

(*a*).—One tension and one bend test shall be made from each melt; except that if material from one melt differs ⅜ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*).—If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*).—If the percentage of elongation of any tension-test specimen is less than that specified in Article 706(*a*) and any part of the fracture is more than ¾ in. from the center of the gauge length of a 2-in. specimen, or is outside the middle third of the gauge length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

IV.—Permissible Variations in Weight and Thickness

711.—The cross-section or weight of each piece of steel shall not vary more than 2.5% from that specified, except in the case of sheared plates which shall be covered by the following permissible variations (1 cu. in. of rolled steel is assumed to weigh 0.2833 lb.):

(*a*).—When ordered to weight per square foot: The weight of each lot† in each shipment shall not vary from the weight ordered more than the amount given in Table 2.

(*b*).—When ordered to thickness: The thickness of each plate shall not vary more than 0.01 in. less than that ordered.

The overweight of each lot‡ in each shipment shall not exceed the amount given in Table 3.

V.—Finish

712.—The finished material shall be free from injurious defects and shall have a workmanlike finish.

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* The gauge length, parallel portions, and fillets shall be as shown in Fig. 5, but the ends may be of any form which will fit the holders of the testing machine.

† The term "lot" applied to Table 2 means all the plates of each group width and group weight.

‡ The term "lot" applied to Table 3 means all the plates of each group width and group thickness.

TABLE 2.—PERMISSIBLE VARIATIONS OF PLATES ORDERED TO WEIGHT.*

Ordered weight, in pounds per square foot.	Permissible Variations in Average Weights per Square Foot of Plates for Widths Given, Expressed in Percentages of Ordered Weights:																	
	Under 48 in.		48 to 60 in., excl.		60 to 72 in., excl.		72 to 84 in., excl.		84 to 96 in., excl.		96 to 108 in., excl.		108 to 120 in., excl.		120 to 132 in., excl.		132 in., or over.	
	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under
Under 5.	5	3	5.5	3	6	3	7	3	....	....	....	....	....	....	....	....	....	....
5 to 7.5, excl.	4.5	3	5	3	5.5	3	6	3	....	....	....	....	....	....	....	....	....	....
7.5 " 10, "	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	....	....	....	....
10 " 12.5, "	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	9	3
12.5 " 15, "	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3
15 " 17.5, "	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3
17.5 " 20, "	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3
20 " 25, "	2	2	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3
25 " 30, "	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3	5	3
30 " 40, "	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3
40, or over	2	2	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3

* The weight per square foot of individual plates shall not vary from the ordered weight by more than one and one-third times the amount given in Table 2.

TABLE 3.—PERMISSIBLE OVERWEIGHTS OF PLATES ORDERED TO THICKNESS.

Ordered thickness, in inches.	Permissible Excess in Average Weights per Square Foot of Plates for Widths Given, Expressed in Percentages of Nominal Weights:								
	Under 48 in.	48 to 60 in., excl.	60 to 72 in., excl.	72 to 84 in., excl.	84 to 96 in., excl.	96 to 108 in., excl.	108 to 120 in., excl.	120 to 132 in., excl.	132 in. or over.
Under 1/8	9	10	12	14	....	....	....	....	....
1/8 to 3/16 excl.	8	9	10	12	....	....	....	....	....
3/16 " 1/4 "	7	8	9	10	12	....	....	....	....
1/4 " 5/16 "	6	7	8	9	10	12	14	16	19
5/16 " 3/8 "	5	6	7	8	9	10	12	14	17
3/8 " 7/16 "	4.5	5	6	7	8	9	10	12	15
7/16 " 1/2 "	4	4.5	5	6	7	8	9	10	13
1/2 " 5/8 "	3.5	4	4.5	5	6	7	8	9	11
5/8 " 3/4 "	3	3.5	4	4.5	5	6	7	8	9
3/4 " 1 "	2.5	3	3.5	4	4.5	5	6	7	8
1, or over	2.5	2.5	3	3.5	4	4.5	5	6	7

VI.—Marking

713.—The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that rivet and lattice-bars and other small sections, when loaded for shipment, shall be properly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

VII.—Inspection and Rejection

714.—Inspection.—The Inspector representing the purchaser shall have free entry at all times, while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manu-

facture of the material ordered. The manufacturer shall afford the Inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be conducted so as not to interfere unnecessarily with the operation of the works.

715.—Rejection.—

(*a*).—Unless otherwise specified, any rejection based on tests made in accordance with Article 705 shall be reported within five working days from the receipt of samples.

(*b*).—Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

716.—Rehearing.—Samples tested in accordance with Article 705, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

Section 8.—Structural Nickel Steel

These specifications conform to the Standard Specifications for Structural Nickel Steel of the American Society for Testing Materials.

I.—Manufacture

801.—Process.—The steel shall be made by the open-hearth process.

802.—Discard.—A sufficient discard shall be made from each ingot intended for eye-bars to secure freedom from injurious piping and undue segregation.

II.—Chemical Properties and Tests

803.—Chemical Composition.—The steel shall conform to the following requirements as to chemical composition:

	Structural steel.	Rivet steel.
Carbon	not over 0.45%	not over 0.30%
Manganese	" " 0.70%	" " 0.60%
Phosphorus, Acid	" " 0.05%	" " 0.04%
Phosphorus, Basic	" " 0.04%	" " 0.03%
Sulphur	" " 0.05%	" " 0.045%
Nickel	not under 3.25%	not under 3.25%

804.—Ladle Analyses.—An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of the elements specified in Article 803. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus determined shall be reported to the purchaser or his representative, and shall conform to the requirements specified in Article 803.

805.—Check Analyses.—Analyses may be made by the purchaser from finished material representing each melt. The chemical composition thus determined shall conform to the requirements specified in Article 803.

III.—Physical Properties and Tests

806.—Tension Tests.—

(*a*).—The material shall conform to the requirements as to tensile properties given in Table 4.

(*b*).—The yield point shall be determined by the drop of the beam of the testing machine.

TABLE 4.

Properties considered.	Rivet steel.	Plates, shapes, and bars.	Eye-bar flats and rollers, ‡ unannealed.	Eye-bar flats* and plus,‡ annealed.
Tensile strength, in pounds per square inch	70 000–80 000	85 000–100 000	95 000–110 000	90 000–105 000
Yield point, minimum, in pounds per square inch	45 000	50 000	55 000	52 000
Elongation in 8 in., minimum, percentage	1 500 000 / Tensile strength	1 500 000† / Tensile strength	1 500 000† / Tensile strength	20
Elongation in 2 in., minimum, percentage	.....	.....	16	20
Reduction of area, minimum, percentage	40	25	25	35

*Tests of annealed specimens of eye-bar flats shall be made for information only.
†See Article 807.
‡Elongation shall be measured in 2 in.

807.—Modifications in Elongation.—For plates, shapes, and unannealed bars of more than 1 in. in thickness, a deduction from the percentage of elongation specified in Article 806(*a*), of 0.25% shall be made for each increase of $\frac{1}{32}$ in. of the specified thickness above 1 in., to a minimum of 14 per cent.

808.—Character of Fracture.—All broken tension-test specimens shall show either a silky or a very fine granular fracture, of uniform color, and free from coarse crystals.

807.—Bend Tests.—

(*a*).—The test specimen for plates, shapes, and bars shall bend cold through 180° without cracking on the outside of the bent portion, as follows: For material $\frac{3}{4}$ in. or less in thickness, around a pin the diameter of which is equal to the thickness of the specimen; and for material more than $\frac{3}{4}$ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

(*b*).—The test specimen for pins and rollers shall bend cold through 180° around a pin 1 in. in diameter, without cracking on the outside of the bent portion.

(*c*).—The test specimen for rivet steel shall bend cold through 180°, flat on itself, without cracking on the outside of the bent portion.

810.—Drift Tests.—Punched rivet holes pitched two diameters from a planed edge shall stand drifting until the diameter is enlarged 50%, without cracking the metal.

811.—Test Specimens.—

(*a*).—Test specimens shall be prepared for testing from the material in its rolled or forged condition, except when it is specified to be annealed, in which case, the test specimens shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated.

(*b*).—Test specimens shall be taken longitudinally and, except as specified in Paragraphs (*d*), (*e*), and (*f*), shall be of the full thickness or diameter of material as rolled.

(*c*).—Test specimens for plates, shapes, and flats may be machined to the form and dimensions shown in Fig. 4, or with both edges parallel; except that bend-test specimens for eye-bar flats may have three-rolled sides.

(*d*).—Tension-test specimens for plates and eye-bar flats more than $1\frac{1}{2}$ in. in thickness, and bend-test specimens for plates more than $1\frac{1}{2}$ in. in thickness, may be machined to a thickness or diameter of at least $\frac{3}{4}$ in. for a length of at least 9 in.

(*e*).—Test specimens for bars more than $1\frac{1}{2}$ in. in thickness or diameter may be machined to a thickness or diameter of at least $\frac{3}{4}$ in. for a length of

at least 9 in.; or tension-test specimens may conform to the dimensions shown in Fig. 5, in which case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens may be 1 in. by ½ in. in section.

(*f*).—Tension-test specimens for pins and rollers shall conform to the dimensions shown in Fig. 5. In this case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens shall be 1 in. by ½ in. in section.

(*g*).—The tension-test specimen shown in Fig. 5 and the 1 in. by ½-in. bend-test specimen for pins and rollers shall be taken so that the axis is 1 in. from the surface; and for other bars more than 1½ in. in thickness or diameter, midway between the center and surface.

(*h*).—The machined sides of rectangular bend-test specimens may have the corners rounded to a radius of not more than $\frac{1}{8}$ in.

(*i*).—Test specimens for rivet bars which have been cold drawn, shall be normalized before testing.

812.—Number of Tests.—

(*a*).—One tension and one bend test shall be made from each melt; except that if material from one melt differs ¾ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

(*b*).—If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

(*c*).—If the percentage of elongation of any tension-test specimen is less than that specified in Article 806(*a*) and any part of the fracture is more than ¾ in. from the center of the gauge length of a 2-in. specimen, or is outside the middle-third of the gauge length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

IV.—Permissible Variations in Weight and Thickness

813.—The cross-section or weight of each piece of steel shall not vary more than 2.5% from that specified; except in the case of sheared plates, which shall be covered by the following permissible variations (1 cu. in. of rolled steel is assumed to weigh 0.2833 lb.):

(*a*).—When ordered to weight per square foot: The weight of each lot* in each shipment shall not vary from the weight ordered more than the amount given in Table 2.

(*b*).—When ordered to thickness: The thickness of each plate shall not vary more than 0.01 in. under that ordered.

The overweight of each lot† in each shipment shall not exceed the amount given in Table 3.

V.—Finish

814.—The finished material shall be free from injurious defects and shall have a workmanlike finish.

VI.—Marking

815.—The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that rivet and lattice-bars and other small sections, when loaded for shipment, shall be prop-

* The term "lot" applied to Table 2 means all the plates of each group width and group weight.

† The term "lot" applied to Table 3 means all the plates of each group width and group thickness.

erly separated and marked for identification. The identification marks shall be legibly stamped on the end of each pin and roller. The melt number shall be legibly marked, by stamping if practicable, on each test specimen.

VII.—Inspection and Rejection

816.—Inspection.—The Inspector representing the purchaser shall have free entry at all times, while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the Inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be conducted so as not to interfere unnecessarily with the operation of the works.

817.—Rejection.—

(*a*).—Unless otherwise specified, any rejection based on tests made in accordance with Article 805 shall be reported within five working days from the receipt of samples.

(*b*).—Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

818.—Rehearing.—Samples tested in accordance with Article 805, which represent rejected material, shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

VIII.—Full-Size Tests

819.—Tests of Eye-bars.—

(*a*).—Full-size tests of annealed eye-bars shall conform to the following requirements as to tensile properties:

Tensile strength, in pounds per square inch...	85 000—100 000
Yield point, minimum, in pounds per square inch	48 000
Elongation in 18 ft., minimum, percentage....	10
Reduction of area, " "	30

(*b*).—The yield point shall be determined by the halt of the gauge of the testing machine.

APPENDIX

Tables 5 to 22, inclusive, and Fig. 6, are submitted in this Appendix, in order to facilitate the use of the Specifications. Because of the fact that two engine loadings are presented, it was thought desirable to furnish working tables for both loadings and a table of coefficients for use in converting E-rating to equivalent M-rating, and *vice versa*.

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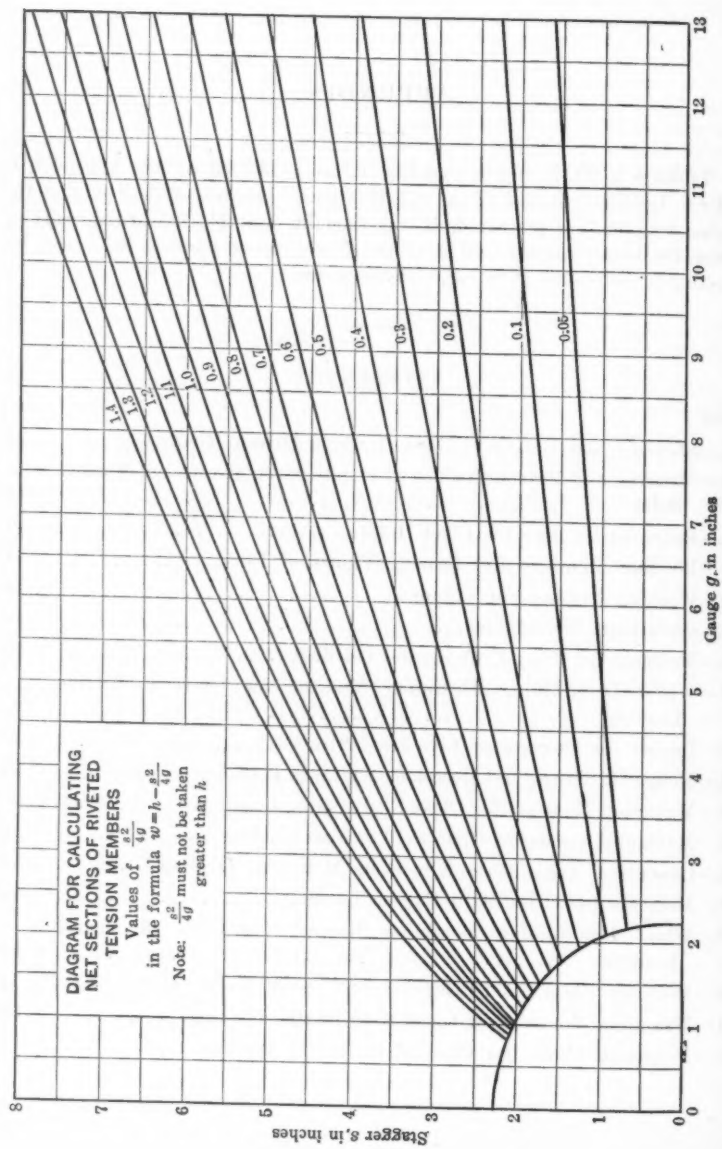

DIAGRAM FOR CALCULATING
NET SECTIONS OF RIVETED
TENSION MEMBERS
Values of $\frac{s^2}{4g}$
in the formula $w = h - \frac{s^2}{4g}$
Note: $\frac{s^2}{4g}$ must not be taken
greater than h
1.4
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.05
Stagger s, in inches
Gauge g, in inches

FIG. 6.

TABLE 5.—Shearing and Bearing Values of Power-Driven Rivets.

Diameter of Rivet.		Area, in square inches.	Single shear @ 12.0 k.	Double shear @ 12.0 k.	Bearing Values at 24.0 k. per sq. in. for Different Thicknesses of Plate.												
Fraction.	Decimal.				¼	5/16	⅜	7/16	½	9/16	⅝	11/16	¾	13/16	⅞	15/16	1
½	0.500	0.1964	2.36	4.71	3.00	3.75	4.50	5.25	....	.....	.....	.....	.....	.....	.....	.....	.....
5/8	0.625	0.3068	3.68	7.36	3.75	4.69	5.63	6.56	7.50	8.44	.....	.....	.....	.....	.....	.....	.....
¾	0.750	0.4418	5.30	10.60	4.50	5.63	6.75	7.88	9.00	10.13	11.25	.....	.....	.....	.....	.....	.....
⅞	0.875	0.6013	7.22	14.43	5.25	6.56	7.88	9.19	10.50	11.81	13.13	14.44	15.75	.....	.....	.....	.....
1	1.000	0.7854	9.43	18.85	6.00	7.50	9.00	10.50	12.00	13.50	15.00	16.50	18.00	19.50	21.00	.....	.....
1⅛	1.125	0.9940	11.93	23.86	6.75	8.44	10.12	11.81	13.50	15.19	16.88	18.56	20.25	21.94	23.62	25.31	27.00
1¼	1.250	1.2272	14.73	29.45	7.50	9.38	11.25	13.12	15.00	16.88	18.75	20.62	22.50	24.38	26.25	28.12	30.00

TABLE 6.—Shearing and Bearing Values of Hand-Driven Rivets and Turned Bolts.

Diameter of Rivet.		Area, in square inches.	Single shear @ 10.0 k.	Double shear @ 10.0 k.	Bearing Values at 20.0 k. per sq. in. for Different Thicknesses of Plate.												
Fraction.	Decimal.				¼	5/16	⅜	7/16	½	9/16	⅝	11/16	¾	13/16	⅞	15/16	1
½	0.500	0.1964	1.96	3.93	2.50	3.13	3.75	4.38	....	.....	.....	.....	.....	.....	.....	.....	.....
5/8	0.625	0.3068	3.07	6.14	3.13	3.91	4.69	5.47	6.25	7.03	.....	.....	.....	.....	.....	.....	.....
¾	0.750	0.4418	4.42	8.84	3.75	4.69	5.63	6.56	7.50	8.44	9.38	.....	.....	.....	.....	.....	.....
⅞	0.875	0.6013	6.01	12.03	4.38	5.47	6.56	7.66	8.75	9.84	10.94	12.03	13.13	.....	.....	.....	.....
1	1.000	0.7854	7.85	15.71	5.00	6.25	7.50	8.75	10.00	11.25	12.50	13.75	15.00	16.25	17.50	.....	.....
1⅛	1.125	0.9940	9.94	19.88	5.63	7.03	8.44	9.85	11.25	12.65	14.06	15.47	16.88	18.28	19.69	21.09	.....
1¼	1.250	1.2272	12.27	24.54	6.25	7.81	9.38	10.94	12.50	14.06	15.62	17.19	18.75	20.31	21.88	23.44	25.00

Note.—Values above or to the right of upper zigzag lines are greater than double shear. Values below or to the left of lower zigzag lines are less than single shear.

TABLE 7.—Reduction of Area for Rivet Holes.

Area in Square Inches = Diameter of Hole by Thickness of Metal.

Thickness of metal, in inches.	Diameter of Hole, in Inches.						
	5/8	3/4	7/8	1.0	1 1/8	1 1/4	1 3/8
3/16	0.12	0.14	0.16	0.19	0.21	0.23	0.26
1/4	0.16	0.19	0.22	0.25	0.28	0.31	0.34
5/16	0.20	0.23	0.27	0.31	0.35	0.39	0.43
3/8	0.23	0.28	0.33	0.38	0.42	0.47	0.52
7/16	0.27	0.33	0.38	0.44	0.49	0.55	0.60
1/2	0.31	0.38	0.44	0.50	0.56	0.63	0.69
9/16	0.35	0.42	0.49	0.56	0.63	0.70	0.77
5/8	0.39	0.47	0.55	0.63	0.70	0.78	0.86
11/16	0.43	0.52	0.60	0.69	0.77	0.86	0.95
3/4	0.47	0.56	0.66	0.75	0.84	0.94	1.03
13/16	0.51	0.61	0.71	0.81	0.91	1.02	1.12
7/8	0.55	0.66	0.77	0.88	0.98	1.09	1.20
15/16	0.59	0.70	0.82	0.94	1.05	1.17	1.29
1.0	0.63	0.75	0.88	1.00	1.13	1.25	1.38
1 1/16	0.66	0.80	0.93	1.06	1.20	1.33	1.46
1 1/8	0.70	0.84	0.98	1.13	1.27	1.41	1.55
1 3/16	0.74	0.89	1.04	1.19	1.34	1.48	1.63
1 1/4	0.78	0.94	1.09	1.25	1.41	1.56	1.72
1 5/16	0.82	0.98	1.15	1.31	1.48	1.64	1.80
1 3/8	0.86	1.03	1.20	1.38	1.55	1.72	1.89
1 7/16	0.90	1.08	1.26	1.44	1.62	1.80	1.98
1 1/2	0.94	1.13	1.31	1.50	1.69	1.88	2.06

TABLE 8.—Bending Moments and Bearing Values of Pins at 24.0 k. per sq. in.

Diameter, in inches.	Area, in square inches.	Maximum moment, in inch-kips.	Bearing value per linear inch, in kips.	Diameter, in inches.	Area, in square inches.	Maximum moment, in inch-kips.	Bearing value per linear inch, in kips.
1	0.785	2.4	24.0	7	38.485	808.2	168.0
1 1/4	1.227	4.6	30.0	7 1/4	41.282	897.9	174.0
1 1/2	1.767	8.0	36.0	7 1/2	44.179	994.0	180.0
1 3/4	2.405	12.6	42.0	7 3/4	47.173	1 096.8	186.0
2	3.142	18.8	48.0	8	50.265	1 206.4	192.0
2 1/4	3.976	26.8	54.0	8 1/4	53.456	1 323.0	198.0
2 1/2	4.909	36.8	60.0	8 1/2	56.745	1 447.0	204.0
2 3/4	5.940	49.0	66.0	8 3/4	60.132	1 578.5	210.0
3	7.069	63.6	72.0	9	63.617	1 717.7	216.0
3 1/4	8.296	80.9	78.0	9 1/4	67.201	1 864.8	222.0
3 1/2	9.621	101.0	84.0	9 1/2	70.882	2 020.1	228.0
3 3/4	11.045	124.3	90.0	9 3/4	74.662	2 183.9	234.0
4	12.566	150.8	96.0	10	78.540	2 356.2	240.0
4 1/4	14.186	180.9	102.0	10 1/4	82.516	2 537.4	246.0
4 1/2	15.904	214.7	108.0	10 1/2	86.590	2 727.6	252.0
4 3/4	17.721	252.5	114.0	10 3/4	90.763	2 927.1	258.0
5	19.635	294.5	120.0	11	95.033	3 136.1	264.0
5 1/4	21.648	340.9	126.0	11 1/4	99.402	3 354.8	270.0
5 1/2	23.758	392.0	132.0	11 1/2	103.869	3 583.5	276.0
5 3/4	25.967	447.9	138.0	11 3/4	108.434	3 822.3	282.0
6	28.274	508.9	144.0	12	113.097	4 071.5	288.0
6 1/4	30.680	575.2	150.0	12 1/4	117.858	4 331.3	294.0
6 1/2	33.183	647.1	156.0	12 1/2	122.718	4 601.9	300.0
6 3/4	35.785	724.6	162.0	12 3/4	127.677	4 883.6	306.0

Note.—1 k. = 1 000 lb.

TABLE 9.—Working Stresses for Columns.

$$p = \frac{16.0\,k}{1 + \frac{l^2}{13\,500\,r^2}}$$

$\frac{l}{r}$	p	$\frac{l}{r}$	p	$\frac{l}{r}$	p	$\frac{l}{r}$	p	$\frac{l}{r}$	p
40	14.30 k	68	11.92 k	96	9.51 k	124	7.48 k	185	4.53 k
42	14.15 "	70	11.74 "	98	9.35 "	126	7.35 "	190	4.35 "
44	13.99 "	72	11.56 "	100	9.19 "	128	7.23 "	195	4.19 "
46	13.83 "	74	11.38 "	102	9.04 "	130	7.11 "	200	4.04 "
48	13.67 "	76	11.21 "	104	8.88 "	135	6.81 "	205	3.89 "
50	13.50 "	78	11.03 "	106	8.73 "	140	6.52 "	210	3.75 "
52	13.33 "	80	10.86 "	108	8.58 "	145	6.25 "	215	3.62 "
54	13.16 "	82	10.68 "	110	8.43 "	150	6.00 "	220	3.49 "
56	12.98 "	84	10.51 "	112	8.29 "	155	5.76 "	225	3.37 "
58	12.81 "	86	10.34 "	114	8.15 "	160	5.52 "	230	3.25 "
60	12.63 "	88	10.17 "	116	8.01 "	165	5.30 "	235	3.14 "
62	12.45 "	90	10.00 "	118	7.87 "	170	5.09 "	240	3.04 "
64	12.27 "	92	9.83 "	120	7.74 "	175	4.89 "	245	2.94 "
66	12.10 "	94	9.67 "	122	7.61 "	180	4.71 "	250	2.84 "

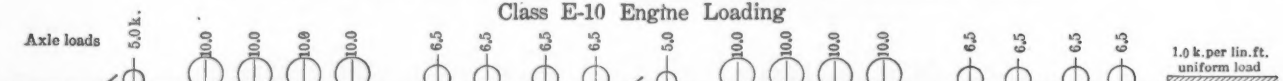
Note.—1 k. = 1 000 lb.

TABLE 10.—Values of Coefficient $\left(1 + \frac{l^2}{13\,500\,r^2}\right)$ to Deduce Extreme Fiber Stress in Columns, from the Direct Applied Stress.

$\frac{l}{r}$	Coefficient.	$\frac{l}{r}$	Coefficient.	$\frac{l}{r}$	Coefficient.	$\frac{l}{r}$	Coefficient.	$\frac{l}{r}$	Coefficient.
40	1.118	68	1.343	96	1.683	124	2.139	185	3.535
42	1.131	70	1.363	98	1.711	126	2.176	190	3.674
44	1.143	72	1.384	100	1.741	128	2.214	195	3.817
46	1.157	74	1.406	102	1.771	130	2.252	200	3.963
48	1.171	76	1.428	104	1.801	135	2.350	205	4.113
50	1.185	78	1.451	106	1.832	140	2.452	210	4.267
52	1.200	80	1.474	108	1.864	145	2.558	215	4.424
54	1.216	82	1.498	110	1.896	150	2.667	220	4.585
56	1.232	84	1.523	112	1.929	155	2.780	225	4.750
58	1.249	86	1.548	114	1.963	160	2.896	230	4.919
60	1.267	88	1.574	116	1.997	165	3.017	235	5.091
62	1.285	90	1.600	118	2.032	170	3.141	240	5.267
64	1.303	92	1.627	120	2.067	175	3.269	245	5.446
66	1.323	94	1.655	122	2.103	180	3.400	250	5.630

TABLE 11.—MOMENTS, IN FOOT-KIPS, FOR CLASS E-10 ENGINE LOADING, ONE TRACK.

Class E-10 Engine Loading



Summations																			
Kips	5.0	15.0	25.0	35.0	45.0	51.5	58.0	64.5	71.0	76.0	86.0	96.0	106.0	116.0	122.5	129.0	135.5	142.0	142.0
Feet	0	8	13	18	23	32	37	43	48	56	64	69	74	79	88	93	99	104	109
Kips	142.0	137.0	127.0	117.0	107.0	97.0	90.5	84.0	77.5	71.0	66.0	56.0	46.0	36.0	26.0	19.5	13.0	6.5	0
Feet	109	101	96	91	86	77	72	66	61	53	45	40	35	30	21	16	10	5	0

Moments of Wheel Loads About Wheel Number

End of Train	8182.0	7637.0	6627.0	5667.0	4787.0	3897.0	3396.5	2928.5	2499.5	2103.0	1838.0	1388.0	988.0	638.0	338.0	201.5	97.5	32.5
18	7472.0	6952.0	5992.0	5082.1	4222.0	3412.0	2944.0	2508.5	2112.0	1748.0	1508.0	1108.0	758.0	458.0	208.0	104.2	32.5	
17	6794.5	6299.5	5389.2	4529.5	3719.5	2959.5	2524.0	2121.0	1757.0	1425.5	1210.5	860.5	560.5	310.5	110.5	39.0	32.5	
16	6020.5	5555.6	4705.5	3905.5	3155.6	2455.5	2059.0	1695.0	1370.0	1077.5	892.5	602.5	362.5	172.5	32.5	39.0	110.5	
15	5408.0	4968.0	4168.0	3418.0	2718.0	2068.0	1704.0	1372.5	1080.0	820.0	660.0	420.0	230.0	90.0	32.5	104.0	208.0	
14	4364.0	3969.0	3259.0	2599.0	1989.0	1429.0	1123.5	850.5	616.5	415.0	300.0	150.0	50.0	58.5	149.5	279.5	442.0	
13	3834.0	3464.0	2804.0	2194.0	1634.0	1124.0	851.0	610.5	409.0	240.0	150.0	50.0	50.0	141.0	264.5	427.0	622.0	
12	3354.0	3009.0	2399.0	1839.0	1329.0	869.0	628.5	420.5	251.5	115.0	50.0	50.0	150.0	273.5	429.5	624.5	852.0	
11	2924.0	2604.0	2044.0	1534.0	1074.0	664.0	456.0	280.5	144.0	40.0	50.0	150.0	300.0	456.0	644.5	872.0	1132.0	
10	2316.0	2036.0	1556.0	1126.0	746.0	416.0	260.0	136.5	52.0	80.0	210.0	390.0	620.0	828.0	1068.5	1348.0	1660.0	
9	1748.0	1508.0	1108.0	758.0	458.0	208.0	104.0	32.5	40.0	200.0	410.0	670.0	980.0	1240.0	1532.5	1864.0	2228.0	
8	1425.5	1210.5	860.5	560.5	310.5	110.5	39.0	32.5	97.5	307.5	567.5	877.5	1237.5	1530.0	1855.0	2219.0	2615.5	
7	1077.2	892.5	602.5	362.5	172.5	32.5	39.0	110.5	205.5	475.5	795.5	1165.5	1585.5	1917.0	2281.0	2684.0	3119.5	
6	820.0	660.0	420.0	230.0	90.0	32.5	104.0	208.0	328.0	648.0	1018.0	1438.0	1908.0	2272.0	2668.5	3104.0	3572.0	
5	415.0	300.0	150.0	50.0	58.5	149.5	279.5	442.0	607.0	1017.0	1477.0	1987.0	2547.0	2969.5	3424.5	3918.5	4445.0	
4	240.0	150.0	50.0	50.0	141.0	264.5	427.0	622.0	812.0	1272.0	1782.0	2342.0	2952.0	3407.0	3894.5	4421.0	4980.0	
3	115.0	50.0	50.0	150.0	273.5	429.5	624.5	852.0	1067.0	1577.0	2137.0	2747.0	3407.0	3894.5	4414.4	4973.5	5565.0	
2	40.0	50.0	150.0	300.0	456.0	644.5	872.0	1132.0	1372.0	1932.0	2542.0	3202.0	3912.0	4482.0	4984.5	5576.0	6200.0	
1	80.0	210.0	390.0	620.0	828.0	1068.5	1348.0	1660.0	1940.0	2580.0	3270.0	4010.0	4800.0	5372.0	5976.5	6620.0	7296.0	

TABLE 12.—Wheel Determining Maximum Moment for Class E Engine Loading.

Segments in feet	10	15	20	25	30	35	40	45	50	55	60	65	70	80	90	100	110	120	130	140
500 to 260	2	3	3	4	4	5	5	6	7	7	8	9	10	11	12	13	14	15	17	18
250 to 200	2	3	3	4	4	5	5	6	7	8	8	9	10	11	12	13	14	15	17	18
190 to 150	2	3	3	4	4	5	5	6	7	8	9	9	11	12	12	13	14	15	17	18
140	3	3	3	4	4	5	5	6	7	8	9	10	11	12	12	13	14	15	17	18
130	3	3	3	4	4	5	5	6	7	8	9	10	11	12	12	13	14	15	17	
120	3	3	3	4	4	5	6	6	7	8	9	10	11	12	13	13	14	15		
110	3	3	3	4	4	5	6	7	7	8	9	10	11	12	13	13	14			
100	3	3	3	4	5	5	$\overline{14}$	$\overline{14}$	$\overline{14}$	$\overline{14}$	$\overline{13}$	$\overline{13}$	11	12	13	13				
90	3	3	4	4	5	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{13}$	12	13					
80	3	3	4	4	$\overline{13}$	$\overline{13}$	$\overline{13}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	12						
70	3	3	4	4	$\overline{13}$	$\overline{13}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{11}$	$\overline{11}$	11							
65	3	3	4	4	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{12}$	$\overline{11}$	$\overline{11}$	11								
60	3	3	4	4	$\overline{13}$	$\overline{13}$	$\overline{12}$	$\overline{11}$	$\overline{11}$	$\overline{11}$	11									
55	12	12	12	4	12	$\overline{13}$	$\overline{12}$	$\overline{12}$	$\overline{13}$	11										
50	12	12	12	12	12	$\overline{13}$	$\overline{13}$	$\overline{13}$	12											
45	12	12	12	12	12	$\overline{13}$	$\overline{13}$	13												
40	3	3	3	12	12	$\overline{13}$	13													
35	3	3	4	4	13	13														
30	3	3	4	4	13															
25	3	3	4	4																
20	$\overline{4}$	3	4																	
15	3	3																		
10	3																			

The shorter segment is ahead followed by the longer one except where the wheel is overlined

Class E Engine

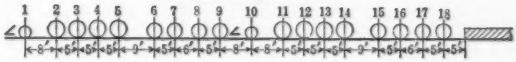


TABLE 13.—IMPACT FOR INCREASE OF LIVE LOAD STRESSES.

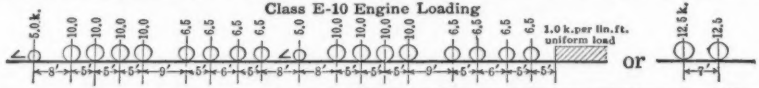
Formula: $I = S \dfrac{2\,000 - L}{1\,600 + 10\,L}$

L.	I.	L.	I.	L.	I.	L.	I.	L.	I.	L.	I.
1	1.24	25	1.07	58	0.89	115	0.69	235	0.45	410	0.28
2	1.23	26	1.06	60	0.88	120	0.67	240	0.44	420	0.27
3	1.23	27	1.06	62	0.87	125	0.66	245	0.43	430	0.27
4	1.22	28	1.05	64	0.86	130	0.64	250	0.43	440	0.26
5	1.21	29	1.04	66	0.86	135	0.63	255	0.42	450	0.25
6	1.20	30	1.04	68	0.85	140	0.62	260	0.41	460	0.25
7	1.19	31	1.03	70	0.84	145	0.61	265	0.41	470	0.24
8	1.19	32	1.03	72	0.83	150	0.60	270	0.40	480	0.24
9	1.18	33	1.02	74	0.82	155	0.59	275	0.40	490	0.23
10	1.17	34	1.01	76	0.82	160	0.58	280	0.39	500	0.23
11	1.16	35	1.01	78	0.81	165	0.56	285	0.39	520	0.22
12	1.16	36	1.00	80	0.80	170	0.55	290	0.38	540	0.21
13	1.15	37	1.00	82	0.79	175	0.54	295	0.37	560	0.20
14	1.14	38	0.99	84	0.79	180	0.54	300	0.37	580	0.19
15	1.13	39	0.99	86	0.78	185	0.53	310	0.36	600	0.18
16	1.13	40	0.98	88	0.77	190	0.52	320	0.35	650	0.17
17	1.12	42	0.97	90	0.76	195	0.51	330	0.34	700	0.15
18	1.11	44	0.96	92	0.76	200	0.50	340	0.33	750	0.14
19	1.11	46	0.95	94	0.75	205	0.49	350	0.32	800	0.13
20	1.10	48	0.94	96	0.74	210	0.48	360	0.32	850	0.11
21	1.09	50	0.93	98	0.74	215	0.48	370	0.31	900	0.10
22	1.09	52	0.92	100	0.73	220	0.47	380	0.30	1 000	0.09
23	1.08	54	0.91	105	0.72	225	0.46	390	0.29	2 000	0.00
24	1.07	56	0.90	110	0.70	230	0.45	400	0.29	.....	....

TABLE 14.—MAXIMUM MOMENTS, SHEARS, AND REACTIONS, CLASS E-10 ENGINE LOADING, ONE TRACK.

TABLE 14.—MAXIMUM MOMENTS, SHEARS, AND REACTIONS, CLASS E-10 ENGINE LOADING, ONE TRACK.

Class E-10 Engine Loading

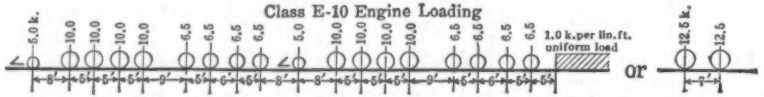


Span, in feet.	Maximum moment, in foot-kips.	Maximum shear, in kips.	Maximum floor-beam reaction, in kips.	Equivalent Uniform Load.			Span, in feet	Maximum moment, in foot-kips.	Maximum shear, in kips.	Maximum floor-beam reaction, in kips.	Equivalent Uniform Load.		
				Moment.	Shear.	Reaction.					Moment.	Shear.	Reaction.
7	21.9	12.5	15.1	3.57	3.57	2.15	42	356.7	39.2	56.0	1.62	1.87	1.34
7½	23.5	13.4	16.0	3.33	3.56	2.12	44	385.8	40.3	58.2	1.60	1.83	1.32
8	25.0	14.0	16.8	3.12	3.51	2.11	46	414.9	41.4	60.3	1.57	1.80	1.31
9	28.1	15.3	18.2	2.78	3.39	2.02	48	443.8	42.4	62.4	1.54	1.77	1.30
10	31.2	16.2	19.2	2.50	3.25	1.92	50	475.5	43.5	64.3	1.52	1.74	1.29
11	34.4	17.0	21.0	2.38	3.09	1.90	52	507.6	44.6	66.7	1.50	1.72	1.28
12	40.0	17.7	23.3	2.22	2.95	1.94	54	540.5	45.6	69.0	1.48	1.69	1.28
13	47.5	18.3	24.6	2.25	2.81	1.90	56	576.1	46.5	71.4	1.47	1.66	1.27
14	55.0	18.8	26.1	2.25	2.68	1.86	58	611.6	47.7	74.0	1.46	1.65	1.28
15	62.5	20.0	27.3	2.22	2.67	1.82	60	649.5	48.8	76.6	1.44	1.63	1.28
16	70.0	21.3	28.5	2.19	2.66	1.78	62	688.2	50.0	79.1	1.43	1.61	1.27
17	77.5	22.4	29.4	2.15	2.63	1.73	64	727.7	51.3	81.5	1.42	1.60	1.27
18	85.0	23.3	30.3	2.10	2.59	1.69	66	769.7	52.5	83.9	1.41	1.59	1.27
19	93.3	24.2	31.5	2.07	2.55	1.66	68	811.7	53.9	86.2	1.40	1.59	1.27
20	103.1	25.0	32.8	2.06	2.50	1.64	70	853.7	55.3	88.5	1.39	1.58	1.26
21	112.9	25.7	34.0	2.05	2.45	1.62	72	896.7	56.7	90.7	1.38	1.58	1.26
22	122.8	26.3	35.1	2.03	2.40	1.60	74	939.0	58.1	93.0	1.37	1.57	1.26
23	132.7	27.0	36.1	2.01	2.34	1.57	76	986.0	59.5	95.2	1.36	1.57	1.25
24	142.6	27.7	37.0	1.98	2.31	1.54	78	1 033.7	60.9	97.3	1.36	1.56	1.25
25	152.5	28.4	37.8	1.95	2.27	1.51	80	1 080.0	62.1	99.4	1.35	1.55	1.24
26	162.4	29.1	38.8	1.92	2.24	1.49	82	1 128.8	63.5	101.5	1.34	1.55	1.24
27	172.3	29.6	40.0	1.89	2.20	1.48	84	1 177.7	64.8	103.5	1.34	1.54	1.23
28	182.7	30.2	41.2	1.86	2.16	1.47	86	1 229.7	66.1	105.4	1.33	1.54	1.23
29	194.0	30.8	42.2	1.84	2.12	1.46	88	1 282.0	67.4	107.3	1.32	1.53	1.22
30	205.2	31.5	43.1	1.82	2.10	1.44	90	1 334.7	68.6	109.3	1.32	1.53	1.22
31	216.5	32.2	44.3	1.80	2.08	1.43	92	1 388.3	69.9	111.2	1.31	1.52	1.21
32	227.7	32.9	45.5	1.78	2.05	1.42	94	1 442.7	71.2	113.1	1.31	1.52	1.20
33	239.0	33.5	46.7	1.75	2.03	1.41	96	1 497.3	72.4	115.0	1.30	1.51	1.20
34	250.3	34.1	47.8	1.73	2.00	1.40	98	1 552.7	73.7	116.8	1.29	1.51	1.19
35	261.5	34.6	48.8	1.71	1.98	1.39	100	1 609.7	75.0	118.6	1.29	1.50	1.19
36	274.3	35.3	49.8	1.69	1.96	1.38	125	2 497.7	89.7	140.5	1.28	1.44	1.12
37	287.2	35.9	50.7	1.68	1.94	1.37	150	3 531.0	103.7	162.7	1.25	1.38	1.08
38	300.0	36.5	51.8	1.66	1.92	1.36	175	4 676.8	117.8	185.8	1.22	1.34	1.06
39	313.3	37.2	52.9	1.65	1.90	1.36	200	5 939.0	130.5	209.5	1.19	1.31	1.05
40	327.8	37.7	54.0	1.64	1.88	1.35	250	8 796.3	156.6	257.6	1.13	1.25	1.03

NOTE.—1 k. = 1 000 lb.

TABLE 15.—MAXIMUM MOMENTS, IN FOOT-KIPS, FOR TRUSS BRIDGES, CLASS E-10 ENGINE LOADING, ONE TRACK.

Class E-10 Engine Loading



PANEL LENGTHS.

Number of panels.	Panel points.	8' 0"	8' 6"	9' 0"	9' 6"	10' 0"	10' 6"	11' 0"	11' 6"	12' 0"	12' 6"	13' 0"
3	½	130	144	157	170	186	201	216	232	248	264	283
4	¼	173	193	212	233	253	275	297	320	344	367	393
	¾	226	250	273	299	328	357	386	415	444	476	506
5	⅕	216	240	265	291	318	344	372	400	428	460	490
	⅖	316	351	386	420	460	502	544	587	630	670	717
6	⅙	256	284	314	344	372	407	438	474	512	553	594
	⅓	403	451	491	539	586	635	687	742	799	857	916
	½	444	488	540	594	647	707	770	833	896	963	1 032
7	⅐	292	325	358	394	432	474	517	564	612	660	710
	⅜	486	538	591	646	703	762	828	901	976	1 057	1 140
	⅞	570	631	696	764	834	908	993	1 077	1 159	1 244	1 333
8	⅛	328	366	408	453	502	550	600	652	706	760	819
	⅜	561	621	684	749	824	909	996	1 083	1 173	1 266	1 362
	⅝	686	760	840	924	1 012	1 107	1 207	1 313	1 425	1 549	1 678
	⅞	728	812	896	985	1 080	1 192	1 281	1 388	1 497	1 608	1 731
9	⅑	368	416	465	515	567	625	679	738	799	860	924
	⅓	633	706	784	872	962	1 057	1 155	1 256	1 360	1 468	1 578
	⅔	799	886	980	1 080	1 194	1 321	1 451	1 579	1 723	1 865	2 007
	⅘	883	990	1 096	1 206	1 317	1 439	1 573	1 716	1 870	2 027	2 189
10	⅒	414	464	518	574	632	693	755	819	885	953	1 023
	⅗	715	804	897	991	1 093	1 200	1 311	1 425	1 541	1 660	1 783
	⅘	910	1 022	1 139	1 270	1 400	1 544	1 695	1 844	1 993	2 144	2 297
	⅚	1 040	1 156	1 285	1 425	1 577	1 739	1 907	2 077	2 256	2 437	2 621
	⅙	1 080	1 202	1 335	1 470	1 610	1 765	1 930	3 110	2 298	2 490	2 687

TABLE 15.—(*Continued.*)

PANEL LENGTHS.

Number of panels.	Panel points.	13′ 6″	14′ 0″	14′ 6″	15′ 0″	15′ 6″	16′ 0″	16′ 6″	17′ 0″	17′ 6″	18′ 0′	18′ 6″
3	½	302	321	340	360	381	403	424	446	468	491	514
4	¼	418	446	473	502	530	560	591	621	652	684	716
	¾	541	576	612	648	688	728	770	812	854	896	940
5	⅕	524	558	594	632	672	715	758	804	849	897	942
	⅖	768	819	871	924	983	1 041	1 099	1 158	1 222	1 285	1 354
6	⅙	640	690	736	786	836	888	941	996	1 050	1 108	1 164
	⅓	980	1 046	1 121	1 196	1 277	1 364	1 451	1 538	1 625	1 723	1 813
	⅚	1 104	1 178	1 255	1 335	1 416	1 497	1 581	1 668	1 767	1 862	1 965
7	⅐	762	819	874	933	992	1 054	1 115	1 178	1 242	1 307	1 374
	⅜	1 224	1 316	1 408	1 504	1 605	1 707	1 808	1 910	2 012	2 113	2 216
	⅝	1 432	1 533	1 644	1 761	1 879	1 999	2 122	2 250	2 382	2 515	2 650
8	⅛	880	943	1 006	1 072	1 138	1 208	1 278	1 349	1 421	1 496	1 572
	⅜	1 466	1 567	1 669	1 774	1 884	1 998	2 112	2 230	2 349	2 472	2 595
	⅝	1 806	1 940	2 076	2 213	2 351	2 491	2 634	2 781	2 931	3 080	3 232
	⅞	1 862	1 999	2 149	2 299	2 452	2 606	2 770	2 932	3 100	3 265	3 442
9	⅑	990	1 060	1 131	1 205	1 278	1 356	1 433	1 514	1 595	1 679	1 764
	⅓	1 690	1 805	1 922	2 043	2 170	2 299	2 430	2 566	2 702	2 843	2 985
	⅔	2 153	2 301	2 451	2 602	2 755	2 909	3 064	3 227	3 400	3 584	3 769
	⅞	2 353	2 526	2 703	2 882	3 063	3 246	3 432	3 619	3 815	4 012	4 213
10	⅒	1 098	1 174	1 253	1 333	1 415	1 500	1 587	1 675	1 766	1 858	1 953
	⅕	1 908	2 037	2 169	2 304	2 445	2 591	2 739	2 891	3 047	3 205	3 367
	⅗	2 451	2 607	2 769	2 946	3 132	3 321	3 514	3 711	3 915	4 125	4 339
	⅘	2 807	2 997	3 194	3 393	3 594	3 798	4 005	4 236	4 477	4 721	4 969
	⅒	2 891	3 098	3 309	3 528	3 751	3 977	4 206	4 438	4 674	4 918	5 166

TABLE 15.—(*Continued*).

PANEL LENGTHS.

Number of panels.	Panel points.	19′ 0″	19′ 6″	20′ 0″	20′ 6″	21′ 0′	21′ 6″	22′ 0″	22′ 6″	23′ 0″	23′ 6″	24′ 0″
3	1/3	538	562	586	....	....	....	....	....	....	....	....
4	1/4	747	783	824	866	909	952	996	1 039	1 083	1 128	1 173
	2/4	986	1 032	1 080	1 128	1 178	1 230	1 282	1 335	1 388	1 443	1 497
5	1/5	991	1 040	1 092	1 146	1 200	1 255	1 311	1 367	1 425	1 482	1 541
	2/5	1 425	1 501	1 577	1 658	1 739	1 822	1 907	1 991	2 077	2 166	2 256
6	1/6	1 225	1 284	1 345	1 406	1 471	1 536	1 603	1 670	1 740	1 809	1 880
	2/6	1 912	2 006	2 104	2 200	2 300	2 399	2 500	2 600	2 702	2 804	2 908
	3/6	2 073	2 185	2 298	2 411	2 528	2 647	2 768	2 891	3 015	3 140	3 266
7	1/7	1 442	1 511	1 582	1 652	1 727	1 802	1 881	1 959	2 040	2 131	2 205
	2/7	2 319	2 422	2 530	2 645	2 766	2 886	3 012	3 138	3 269	3 401	3 537
	3/7	2 785	2 920	3 058	3 196	3 339	3 484	3 632	3 779	3 930	4 086	4 244
8	1/8	1 650	1 728	1 810	1 891	1 976	2 060	2 149	2 239	2 332	2 424	2 520
	2/8	2 722	2 850	2 988	3 122	3 265	3 408	3 556	3 704	3 856	4 012	4 172
	3/8	3 385	3 540	3 694	3 855	4 028	4 211	4 397	4 593	4 790	4 989	5 192
	4/8	3 617	3 796	3 977	4 158	4 345	4 531	4 722	4 918	5 116	5 315	5 518
9	1/9	1 852	1 940	2 032	2 123	2 218	2 312	2 412	2 512	2 617	2 722	2 830
	2/9	3 132	3 279	3 431	3 588	3 751	3 916	4 086	4 256	4 433	4 610	4 794
	3/9	3 957	4 155	4 352	4 555	4 760	4 973	5 191	5 417	5 647	5 882	6 123
	4/9	4 426	4 649	4 869	5 094	5 324	5 555	5 789	6 027	6 274	6 520	6 772
10	1/10	2 049	2 148	2 249	2 352	2 456	2 563	2 672	2 783	2 898	3 015	3 134
	2/10	3 531	3 699	3 871	4 045	4 229	4 417	4 608	4 803	5 000	5 201	5 407
	3/10	4 557	4 778	5 008	5 242	5 480	5 721	5 966	6 228	6 491	6 763	7 040
	4/10	5 220	5 478	5 743	6 011	6 283	6 558	6 837	7 120	7 411	7 707	8 006
	5/10	5 417	5 671	5 928	6 189	6 458	6 731	7 007	7 286	7 568	7 863	8 162

TABLE 15.—(*Continued*).

PANEL LENGTHS.

Number of panels.	Panel points.	24′ 6″	25′ 0″	25′ 6″	26′ 0″	26′ 6″	27′ 0″	27′ 6″	28′ 0″	28′ 6″	29′ 0″	29′ 6″
3	½	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	¼	1 218	1 265	1 313	1 364	1 415	1 466	1 510	1 566	1 621	1 666	1 720
	¾	1 553	1 610	1 667	1 731	1 811	1 862	1 943	1 999	2 072	2 147	2 222
5	⅕	1 600	1 660	1 720	1 782	1 844	1 908	1 972	2 037	2 102	2 169	2 236
	⅖	2 346	2 437	2 530	2 621	2 713	2 807	2 900	2 997	3 094	3 194	3 293
6	⅙	1 951	2 024	2 098	2 178	2 249	2 326	2 404	2 483	2 563	2 645	2 727
	⅔	3 010	3 118	3 227	3 341	3 462	3 584	3 707	3 832	3 959	4 087	4 219
	⅚	3 396	3 528	3 661	3 796	3 931	4 068	4 206	4 345	4 488	4 626	4 770
7	⅐	2 288	2 374	2 460	2 549	2 638	2 729	2 820	2 914	3 008	3 105	3 201
	⅜	3 673	3 812	3 950	4 094	4 240	4 392	4 543	4 697	4 850	5 008	5 167
	⅞	4 410	4 578	4 750	4 925	5 101	5 281	5 461	5 645	5 828	6 016	6 203
8	⅛	2 616	2 715	2 814	2 916	3 016	3 122	3 228	3 335	3 443	3 555	3 666
	⅜	4 333	4 498	4 662	4 832	5 003	5 180	5 357	5 540	5 723	5 912	6 100
	⅝	5 396	5 604	5 814	6 025	6 242	6 465	6 687	6 914	7 141	7 372	7 604
	⅞	5 720	5 928	6 136	6 350	6 565	6 786	7 006	7 229	7 454	7 684	7 925
9	⅑	2 938	3 049	3 160	3 275	3 391	3 510	3 628	3 750	3 874	3 998	4 124
	⅓	4 979	5 170	5 362	5 556	5 752	5 955	6 160	6 372	6 584	6 802	7 019
	⅔	6 364	6 611	6 858	7 111	7 366	7 628	7 892	8 162	8 432	8 708	8 984
	⅞	7 024	7 282	7 542	7 806	8 076	8 348	8 623	8 904	9 185	9 471	9 768
10	⅒	3 256	3 379	3 504	3 631	3 761	3 892	4 025	4 160	4 298	4 437	4 579
	⅕	5 618	5 833	6 050	6 271	6 495	6 722	6 956	7 195	7 436	7 681	7 928
	⅗	7 320	7 603	7 891	8 182	8 474	8 778	9 086	9 397	9 711	10 030	10 352
	⅘	8 309	8 616	8 926	9 240	9 564	9 891	10 222	10 556	10 894	11 237	11 593
	⅒	8 468	8 780	9 095	9 417	9 746	10 078	10 416	10 760	11 108	11 463	11 828

TABLE 15.—(*Continued.*)

PANEL LENGTHS.

Number of panels.	Panel points.	30′ 0″	30′ 6″	31′ 0″	31′ 6″	32′ 0″	32′ 6″	33′ 0″	33′ 6″	34′ 0″	34′ 6″	35′ 0″
3	1/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	1/4	1 774	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/4	2 298	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	1/5	2 304	2 375	2 445	2 518	2 591	2 672	2 740	2 816	2 891	2 969	3 047
	2/5	3 393	3 494	3 594	3 696	3 798	3 900	4 005	4 118	4 236	4 356	4 477
6	1/6	2 810	2 895	2 980	3 068	3 157	3 248	3 339	3 432	3 525	3 620	3 715
	2/6	4 352	4 488	4 623	4 761	4 899	5 044	5 192	5 342	5 492	5 648	5 804
	3/6	4 918	5 067	5 216	5 367	5 518	5 672	5 825	5 981	6 136	6 298	6 459
7	1/7	3 300	3 400	3 501	3 604	3 706	3 814	3 922	4 032	4 142	4 255	4 368
	2/7	5 332	5 499	5 665	5 836	6 006	6 179	6 354	6 528	6 724	6 914	7 103
	3/7	6 394	6 590	6 786	6 986	7 185	7 390	7 592	7 798	8 006	8 218	8 429
8	1/8	3 780	3 896	4 012	4 130	4 249	4 372	4 496	4 623	4 750	4 880	5 010
	2/8	6 292	6 490	6 688	6 891	7 093	7 301	7 508	7 724	7 941	8 163	8 384
	3/8	7 840	8 082	8 324	8 573	8 820	9 074	9 327	9 584	9 840	10 104	10 368
	4/8	8 162	8 409	8 655	8 907	9 159	9 419	9 680	9 946	10 212	10 486	10 760
9	1/9	4 253	4 384	4 515	4 650	4 784	4 924	5 063	5 207	5 351	5 499	5 646
	2/9	7 240	7 469	7 698	7 933	8 168	8 408	8 645	8 896	9 144	9 401	9 658
	3/9	9 267	9 554	9 841	10 137	10 433	10 736	11 038	11 346	11 654	11 969	12 284
	4/9	10 068	10 371	10 676	10 999	11 312	11 638	11 963	12 296	12 629	12 972	13 316
10	1/10	4 709	4 868	5 039	5 165	5 316	5 468	5 625	5 785	5 947	6 111	6 277
	2/10	8 180	8 435	8 696	8 961	9 229	9 500	9 775	10 052	10 336	10 627	10 920
	3/10	10 678	11 007	11 342	11 684	12 030	12 380	12 733	13 089	13 450	13 811	14 183
	4/10	11 952	12 316	12 688	13 065	13 444	13 831	14 226	14 623	15 025	15 435	15 850
	5/10	12 200	12 578	12 963	13 353	13 750	14 154	14 563	14 978	15 400	15 828	16 263

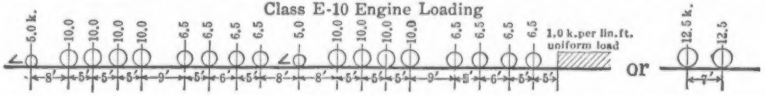
TABLE 15.—(*Continued.*)

PANEL LENGTHS

Number of panels.	Panel points.	35′ 6″	36′ 0″	37′ 0″	38′ 0″	39′ 0″	40′ 0″	41′ 0″	42′ 0″	43′ 0″	44′ 0″	45′ 0″
3	1/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	1/4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	
5	1/5	3 126	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/5	4 598	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
6	1/6	3 851	3 910	4 109	4 312	4 519	4 730	4 945	5 166	5 397	5 632	5 872
	2/6	5 961	6 124	6 448	6 777	7 112	7 454	7 806	8 164	8 526	8 894	9 268
	3/6	6 662	6 787	7 119	7 456	7 803	8 164	8 531	8 906	9 287	9 680	10 079
7	1/7	4 481	4 598	4 834	5 075	5 323	5 572	5 828	6 087	6 361	6 640	6 925
	2/7	7 294	7 496	7 895	8 301	8 715	9 136	9 568	10 010	10 460	10 916	11 380
	3/7	8 642	8 866	9 308	9 757	10 222	10 700	11 187	11 688	12 195	12 720	13 251
8	1/8	5 142	5 277	5 549	5 828	6 113	6 404	6 701	6 998	7 315	7 638	7 968
	2/8	8 607	8 848	9 322	9 805	10 297	10 798	11 310	11 837	12 372	12 917	13 470
	3/8	10 634	10 911	11 463	12 025	12 604	13 201	13 810	14 436	15 070	15 726	16 390
	4/8	11 036	11 319	11 901	12 459	13 117	13 744	*14 397	15 061	15 741	16 437	17 149
9	1/9	5 796	5 948	6 258	6 574	6 898	7 228	7 566	7 911	8 262	8 630	9 004
	2/9	9 918	10 188	10 737	11 296	11 866	12 447	13 039	13 649	14 270	14 903	15 546
	3/9	12 600	12 934	13 596	14 270	14 965	15 681	16 410	17 161	17 923	18 709	19 540
	4/9	13 662	14 015	14 745	15 495	16 265	17 055	17 865	18 695	19 545	20 415	21 305
10	1/10	6 445	6 615	6 961	7 315	7 677	8 048	8 426	8 813	9 207	9 616	10 035
	2/10	11 217	11 519	12 141	12 776	13 424	14 085	14 759	15 450	16 158	16 878	17 610
	3/10	14 560	14 940	15 712	16 499	17 307	18 142	18 992	19 870	20 759	21 676	22 607
	4/10	16 263	16 692	17 568	18 468	19 392	20 340	21 312	22 308	23 328	24 372	25 440
	5/10	16 703	17 150	18 063	19 000	19 963	20 950	21 963	23 000	24 063	25 150	26 263

TABLE 16.—Maximum Shears, in Kips, for Truss Bridges, Class E-10 Engine-Loading, One Track.

Class E-10 Engine Loading



Panel Lengths.

Number of panels.	Panel point.	8′ 0″	8′ 6″	9′ 0″	9′ 6″	10′ 0″	10′ 6″	11′ 0″	11′ 6″	12′ 0″	12′ 6″	13′ 0″
3	1/8	16.2	16.8	17.4	17.9	18.6	19.2	19.7	20.2	20.6	21.2	21.7
	2/8	2.9	3.2	3.5	3.8	4.0	4.4	4.7	5.0	...	...	...
4	1/4	21.6	22.7	23.6	24.5	25.2	26.2	27.0	27.8	28.6	29.4	30.2
	2/4	9.4	10.2	10.8	11.4	12.0	12.5	13.0	13.4	13.8	14.2	14.7
	3/4	1.3	1.4	1.5	1.8	2.0	2.3	2.6	2.9	3.2	3.4	3.6
5	1/5	26.9	28.1	29.4	30.6	31.8	32.8	33.8	34.8	35.7	36.6	37.4
	2/5	15.5	16.4	17.2	18.0	18.7	19.5	20.1	20.8	21.5	22.2	22.8
	3/5	6.5	7.2	7.8	8.3	8.8	9.2	9.6	10.0	10.4	10.8	11.1
6	1/6	32.0	33.4	34.8	36.0	37.4	38.8	40.0	41.2	42.7	44.2	45.7
	2/6	21.1	22.1	23.2	24.2	25.2	26.2	27.1	28.0	28.8	29.7	30.5
	3/6	12.1	12.7	13.6	14.2	15.0	15.6	16.2	16.8	17.4	18.0	18.5
	4/6	4.6	5.2	5.8	6.3	6.6	7.1	7.4	7.8	8.1	8.4	8.8
7	1/7	36.4	38.0	39.7	41.4	43.2	45.1	47.0	49.2	51.0	52.8	54.6
	2/7	26.2	27.6	29.0	30.1	31.3	32.4	33.6	34.4	35.6	36.8	38.0
	3/7	17.4	18.2	19.2	20.2	21.0	21.9	22.8	23.5	23.8	24.8	25.7
	4/7	9.6	10.4	11.0	11.6	12.2	12.8	13.4	13.9	14.4	15.0	15.4
	5/7	.....	.....	.....	.....	.....	.....	.....	.....	6.4	6 8	7.1
8	1/8	40.8	43.0	45.4	47.7	50.2	52.4	54.6	56.8	58.9	60.9	63.0
	2/8	31.3	32.7	34.2	35.6	37.0	38.4	39.9	41.6	43.4	45.0	46.7
	3/8	22.3	23.6	24.8	25.8	26.9	27.8	28.9	29.8	30.7	31.8	32.9
	4/8	14.6	15.4	16.3	17.1	17.8	18.7	19.4	20.2	20.8	21.5	22.1
	5/8	7.8	8.5	9.1	9.6	10,2	10.8	11.2	11.6	12,2	12.7	13,1
	6/8	.....	.....	.....	.....	.....	.....	.....	.....	5.2	5.5	5.8
9	1/9	46.1	48.9	51.7	54.2	56.8	59.2	61.7	64.3	66.6	68,8	71.0
	2/9	35.8	37.4	39.4	41.3	43.3	45.4	47.4	49.4	51.3	53.2	55.0
	3/9	27.2	28.6	29.9	31.0	32.5	33.7	35.1	36.6	38.2	39.7	41.2
	4/9	19.3	20.4	21.5	22.6	23.4	24.3	25.2	26.0	27.0	27.9	28.9
	5/9	12.4	13.2	14.0	14.8	15.4	16.2	16.9	17.5	18.1	18.7	19.3
	6/9	.....	.....	.....	.....	.....	.....	.....	.....	10,5	10.9	11.3
	7/9	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
10	1/10	51.7	54.6	57.6	60.4	63.2	66.0	68.7	71.2	73.8	76.3	78.8
	2/10	40.5	42.9	45.3	47.8	50.0	52.3	54.6	56.8	58.9	61.1	63.2
	3/10	31.5	33.3	35.0	36.7	38.6	40.5	42.4	44.1	45.8	47.5	49.2
	4/10	24.0	25.2	26.4	27.5	28.7	30.0	31.2	32.6	34.0	35.4	36.7
	5/10	16.8	17.9	18.9	19.8	20.7	21.5	22.3	23.1	23.9	24.8	25.7
	6/10	10.6	11.3	12.1	12.8	13.5	14.2	14.8	15.4	16.0	16.6	17.1
	7/10	5.8	6.2	6.6	7.0	7.4	7.8	8.3	8.7	9.1	9.5	9.9
	8/10	2.3	2.5	2.7	2.9	3.1	3.3	3.5	3.7	3.8	4.0	4.2

TABLE 16.—(*Continued.*)

PANEL LENGTHS.

Number of panels.	Panel point.	13′ 6″	14′ 0″	14′ 6″	15′ 0″	15′ 6″	16′ 0″	16′ 6″	17′ 0″	17′ 6″	18′ 0″	18′ 6″
3	1/3	22.4	23.0	23.5	24.0	24.6	25.2	25.7	26.2	26.8	27.3	27.8
	2/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	1/4	31.0	31.8	32.6	33.4	34.2	34.9	35.6	36.3	37.0	37.8	38.6
	2/4	15.1	15.4	15.8	16.2	16.7	17.1	17.6	18.0	18.4	18.9	19.3
	3/4	3.8	3.9	4.1	4.3	4.5	4.7	4.9	5.1	5.2	5.4	5.6
5	1/5	38.6	39.7	40.9	42.2	43.4	44.7	46.0	47.3	48.6	49.8	51.0
	2/5	23.5	24.1	24.8	25.4	25.9	26.5	27.1	27.7	28.3	29.0	29.6
	3/5	11.5	11.8	12.2	12.5	12.8	13.1	13.4	13.7	14.0	14.3	14.6
6	1/6	47.5	49.2	50.8	52.4	54.0	55.5	57.1	58.6	60.1	61.5	63.0
	2/6	31.2	31.9	32.9	33.8	34.8	36.0	37.2	38.3	39.4	40.4	41.4
	3/6	19.1	19.6	20.2	20.7	21.2	21.6	22.1	22.6	23.0	23.4	23.8
	4/6	9.0	9.3	9.6	9.9	10.2	10.5	10.8	11.0	11.3	11.6	11.8
7	1/7	56.6	58.5	60.4	62.2	64.0	65.8	67.6	69.3	71.0	72.6	74.3
	2/7	39.5	41.0	42.4	43.8	45.2	46.6	47.9	49.2	50.5	51.8	53.1
	3/7	26.4	27.0	27.7	28.4	29.2	30.1	31.0	31.9	32.8	33.7	34.6
	4/7	15.9	16.4	16.9	17.4	17.8	18.2	18.6	19.0	19.4	19.8	20.2
	5/7	7.4	7.6	7.9	8.1	8.4	8.6	8.9	9.1	9.4	9.6	9.8
8	1/8	65.2	67.4	69.4	71.5	73.5	75.5	77.4	79.3	81.2	83.1	85.0
	2/8	48.4	50.1	51.8	53.5	55.1	56.7	58.3	59.8	61.3	62.8	64.2
	3/8	34.0	35.1	36.4	37.6	38.7	39.8	41.0	42.2	43.4	44.6	45.7
	4/8	22.7	23.2	23.9	24.6	25.2	25.9	26.6	27.4	28.2	28.9	29.6
	5/8	13.6	14.0	14.4	14.8	15.2	15.6	16.0	16.4	16.7	17.0	17.4
	6/8	6.0	6.3	6.6	6.8	7.0	7.2	7.5	7.7	7.9	8.1	8.3
9	1/9	73.4	75.8	78.0	80.3	82.5	84.7	86.9	89.1	91.2	93.3	95.4
	2/9	57.0	59.0	60.8	62.7	64.5	66.3	68.0	69.8	71.5	73.2	74.9
	3/9	42.6	43.9	45.2	46.7	48.2	49.6	51.0	52.4	53.8	55.1	56.4
	4/9	29.9	30.9	32.0	33.1	34.1	35.0	36.0	37.0	38.0	39.0	40.0
	5/9	19.8	20.3	21.0	21.5	22.2	22.8	23.4	24.1	24.8	25.4	26.1
	6/9	11.7	12.1	12.6	12.9	13.2	13.6	13.9	14.3	14.6	14.9	15.2
	7/9	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
10	1/10	81.3	83.9	86.4	88.9	91.3	93.8	96.2	98.5	100.9	103.3	105.6
	2/10	65.3	67.4	69.5	71.5	73.6	75.6	77.5	79.5	81.4	83.3	85.2
	3/10	50.9	52.6	54.2	55.7	57.3	59.0	60.6	62.1	63.7	65.2	66.8
	4/10	38.0	39.3	40.6	41.8	43.0	44.2	45.4	46.6	47.7	48.9	50.1
	5/10	26.6	27.5	28.5	29.5	30.4	31.3	32.2	33.1	34.0	34.8	35.7
	6/10	17.6	18.1	18.6	19.1	19.7	20.3	20.8	21.4	22.1	22.7	23.3
	7/10	10.2	10.6	11.0	11.3	11.7	12.0	12.3	12.6	12.9	13.3	13.5
	8/10	4.4	4.6	4.7	4.9	5.1	5.3	5.5	5.7	5.9	6.1	6.3

TABLE 16 —(*Continued.*)

PANEL LENGTHS.

Number of panels.	Panel point.	19′ 0″	19′ 6″	20′ 0″	20′ 6″	21′ 0″	21′ 6″	22′ 0″	22′ 6″	23′ 0″	23′ 6″	24′ 0″
3	1/3	28.3	28.8	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	1/4	39.3	40.3	41.2	42.2	43.3	44.3	45.3	46.2	47.1	48.0	48.9
	2/4	19.7	20.1	20.5	20.9	21.2	21.6	22.0	22.3	22.7	23.0	23.3
	3/4	5.7	5.9	6.0	6.1	6.2	6.4	6.5	6.6	6.7	6.8	6.9
5	1/5	52.2	53.4	54.6	55.9	57.2	58.4	59.6	60.8	62.0	63.1	64.2
	2/5	30.3	31.0	31.6	32.4	33.0	33.8	34.4	35.2	36.0	36.7	37.4
	3/5	14.9	15.2	15.5	15.8	16.1	16.4	16.6	16.9	17.2	17.5	17.7
6	1/6	64.4	65.8	67.2	68.7	70.1	71.5	72.9	74.3	75.6	77.0	78.3
	2/6	42.4	43.4	44.4	45.4	46.4	47.4	48.3	49.3	50.2	51.1	52.0
	3/6	24.3	24.8	25.4	26.0	26.6	27.3	27.9	28.5	29.2	29.8	30.4
	4/6	12.1	12.4	12.6	12.8	13.1	13.4	13.6	13.8	14.0	14.2	14.4
7	1/7	75.9	77.6	79.1	80.7	82.2	83.8	85.5	87.2	88.7	90.3	91.9
	2/7	54.4	55.6	56.8	58.0	59.2	60.4	61.5	62.4	63.7	64.8	65.9
	3/7	35.5	36.4	37.2	38.1	39.0	39.8	40.6	41.5	42.3	43.2	43.9
	4/7	20.6	21.0	21.4	21.8	22.2	22.7	23.2	23.7	24.2	24.8	25.4
	5/7	10.0	10.3	10.5	10.8	11.0	11.2	11.4	11.6	11.8	12.0	12.1
8	1/8	86.8	88.7	90.5	92.3	94.1	95.9	97.7	99.6	101.4	103.2	105.0
	2/8	65.6	67.1	68.5	69.9	71.2	72.6	74.0	75.4	76.7	78.0	79.3
	3/8	46.8	47.9	49.0	50.0	51.0	52.1	53.2	54.2	55.1	56.1	57.1
	4/8	30.3	31.1	31.9	32.7	33.4	34.2	34.9	35.6	36.4	37.1	37.8
	5/8	17.7	18.1	18.4	18.8	19.2	19.6	20.0	20.4	20.8	21.2	21.6
	6/8	8.5	8.8	9.0	9.2	9.4	9.6	9.8	10.0	10.1	10.3	10.4
9	1/9	97.4	99.5	101.6	103.6	105.6	107.6	109.6	111.7	113.8	115.9	118.0
	2/9	76.5	78.2	79.8	81.4	83.0	84.6	86.2	87.8	89.3	90.9	92.4
	3/9	57.7	59.0	60.2	61.5	62.8	64.0	65.2	66.4	67.6	68.8	70.0
	4/9	41.0	42.0	42.9	43.9	44.8	45.7	46.6	47.6	48.5	49.4	50.2
	5/9	26.8	27.4	28.0	28.7	29.3	30.0	30.6	31.2	31.8	32.5	33.1
	6/9	15.5	15.8	16.2	16.5	16.8	17.2	17.6	18.0	18.3	18.7	19.0
	7/9	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
10	1/10	107.9	110.1	112.4	114.7	117.0	119.3	121.5	123.8	126.0	128.3	130.6
	2/10	87.1	89.0	90.8	92.7	94.5	96.4	98.2	100.0	101.7	103.5	105.3
	3/10	68.3	69.8	71.2	72.7	74.1	75.6	77.1	78.5	79.9	81.3	82.7
	4/10	51.3	52.5	53.6	54.8	55.9	57.0	58.2	59.3	60.3	61.4	62.5
	5/10	36.5	37.4	38.2	39.0	39.8	40.6	41.4	42.3	43.1	43.9	44.7
	6/10	23.9	24.5	25.0	25.6	26.2	26.8	27.3	27.9	28.4	29.0	29.5
	7/10	13.8	14.1	14.4	14.7	15.0	15.3	15.6	15.9	16.3	16.6	17.0
	8/10	6.4	6.6	6.7	6.9	7.1	7.3	7.4	7.6	7.7	7.9	8.0

TABLE 16.—(*Continued*)

Number of panels.	Panel point.	Panel Lengths. 24' 6"	25' 0"	25' 6"	26' 0"	26' 6"	27' 0"	27' 6"	28' 0"	28' 6"	29' 0"	29' 6"
3	1/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	1/4	49.8	50.6	51.5	52.4	53.2	54.1	54.9	55.7	56.6	57.4	58.3
	2/4	23.6	23.9	24.2	24.5	24.8	25.2	25.5	25.8	26.2	26.6	27.0
	3/4	7.0	7.1	7.2	7.3	7.4	7.5	7.6	7.7	7.8	7.9	8.0
5	1/5	65.3	66.4	67.5	68.6	69.6	70.7	71.8	72.8	73.8	74.8	75.8
	2/5	38.0	38.6	29.3	40.0	40.7	41.4	42.1	42.8	43.5	44.2	44.9
	3/5	18.0	18.2	18.5	18.8	19.1	19.3	19.6	19.9	20.2	20.5	20.8
6	1/6	79.6	81.0	82.3	83.6	84.9	86.2	87.4	88.7	90.0	91.2	92.4
	2/6	52.9	53.8	54.7	55.6	56.5	57.4	58.3	59.2	60.1	61.0	61.8
	3/6	31.0	31.5	32.1	32.6	33.2	33.7	34.3	34.8	35.3	35.8	36.4
	4/6	14.6	14.8	15.0	15.2	15.4	15.6	15.8	16.0	16.2	16.4	16.6
7	1/7	93.4	95.0	96.6	98.1	99.6	101.1	102.6	104.1	105.6	107.1	108.6
	2/7	67.0	68.1	69.3	70.4	71.5	72.6	73.7	74.8	75.9	77.0	78.1
	3/7	44.7	45.4	46.2	47.0	47.7	48.4	49.2	49.9	50.6	51.3	52.0
	4/7	25.9	26.3	26.8	27.3	27.8	28.3	28.8	29.2	29.7	30.2	30.7
	5/7	12.3	12.5	12.7	12.8	13.0	13.2	13.4	13.5	13.7	13.8	14.0
8	1/8	106.8	108.6	110.4	112.2	114.0	115.7	117.4	119.2	120.9	122.6	124.3
	2/8	80.7	82.0	83.3	84.6	86.0	87.4	88.7	90.0	91.4	92.7	94.0
	3/8	58.1	59.0	60.0	60.9	61.9	62.8	63.8	64.7	65.6	66.5	67.4
	4/8	38.5	39.2	39.9	40.6	41.2	41.8	42.5	43.2	43.8	44.4	45.0
	5/8	22.1	22.6	23.0	23.4	23.8	24.2	24.6	25.0	25.5	25.9	26.4
	6/8	10.6	10.8	10.9	11.0	11.2	11.4	11.5	11.6	11.8	12.0	12.2
9	1/9	120.0	122.0	124.0	126.0	128.0	130.0	132.0	134.0	136.0	137.9	139.9
	2/9	94.0	95.5	97.1	98.7	100.2	101.8	103.4	105.0	106.5	108.1	109.6
	3/9	71.2	72.3	73.5	74.7	75.8	77.0	78.1	79.2	80.4	81.5	82.6
	4/9	51.1	52.0	52.8	53.6	54.5	55.4	56.2	57.0	57.8	58.6	59.4
	5/9	33.7	34.3	35.0	35.6	36.2	36.7	37.3	37.9	38.5	39.0	39.6
	6/9	19.4	19.8	20.2	20.6	21.0	21.3	21.7	22.0	22.4	22.7	23.0
	7/9	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
10	1/10	132.9	135.2	137.4	139.7	141.9	144.1	146.4	148.6	150.8	153.0	155.2
	2/10	107.1	108.8	110.7	112.5	114.3	116.1	117.9	119.7	121.5	123.3	125.0
	3/10	84.1	85.5	86.9	88.2	89.6	90.9	92.3	93.7	95.1	96.4	97.8
	4/10	63.6	64.6	65.7	66.7	67.8	68.8	69.9	70.9	71.9	72.9	74.0
	5/10	45.5	46.2	47.0	47.8	48.6	49.4	50.1	50.9	51.7	52.4	53.1
	6/10	30.1	30.6	31.1	31.6	32.1	32.6	33.2	33.7	34.2	34.7	35.3
	7/10	17.3	17.7	18.0	18.3	18.7	19.1	19.4	19.7	20.0	20.3	20.6
	8/10	8.1	8.3	8.4	8.5	8.7	8.8	8.9	9.0	9.2	9.3	9.5

TABLE 16.—(*Continued*).

Number of panels.	Panel point.	Panel Lengths. 30′ 0″	30′ 6″	31′ 0″	31′ 6″	32′ 0″	32′ 6″	33′ 0″	33′ 6″	34′ 0″	34′ 6″	35′ 0″
3	1/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	1/4	59.2	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/4	27.3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	3/4	8.1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	1/5	76.8	77.8	78.8	79.9	81.0	82.0	83.0	84.0	85.0	86.0	87.0
	2/5	45.6	46.2	46.9	47.6	48.2	48.8	49.4	50.0	50.6	51.2	51.8
	3/5	21.1	21.4	21.7	22.0	22.3	22.7	23.0	23.3	23.6	24.0	24.3
6	1/6	93.7	94.9	96.1	97.4	98.6	99.9	101.2	102.4	103.6	104.9	106.2
	2/6	62.7	63.5	64.4	65.2	66.0	66.9	67.7	68.6	69.4	70.2	71.0
	3/6	37.0	37.5	38.0	38.5	39.0	39.5	40.0	40.5	41.0	41.5	42.0
	4/6	16.9	17.2	17.5	17.8	18.0	18.3	18.6	18.9	19.2	19.4	19.7
7	1/7	110.0	111.5	112.9	114.4	115.8	117.3	118.8	120.3	121.8	123.3	124.8
	2/7	79.2	80.2	81.3	82.4	83.4	84.5	85.5	86.6	87.6	88.6	89.6
	3/7	52.7	53.4	54.1	54.8	55.6	56.3	57.0	57.7	58.4	59.1	59.8
	4/7	31.1	31.6	32.0	32.5	33.0	33.4	33.8	34.2	34.6	35.1	35.5
	5/7	14.2	14.4	14.6	14.8	15.0	15.2	15.4	15.7	16.0	16.2	16.4
8	1/8	126.0	127.7	129.4	131.1	132.8	134.5	136.2	138.0	139.7	141.5	143.2
	2/8	95.3	96.6	97.8	99.1	100.4	101.7	103.0	104.2	105.5	106.8	108.0
	3/8	68.3	69.2	70.2	71.1	72.1	73.0	73.9	74.8	75.8	76.7	77.6
	4/8	45.6	46.3	46.9	47.5	48.1	48.8	49.4	50.0	50.6	51.2	51.8
	5/8	26.8	27.2	27.6	28.0	28.4	28.8	29.2	29.6	29.9	30.3	30.6
	6/8	12.3	12.5	12.6	12.8	13.0	13.2	13.3	13.5	13.7	13.9	14.
9	1/9	141.8	143.7	145.6	147.6	149.6	151.5	153.4	155.4	157.4	159.4	161.3
	2/9	111.1	112.6	114.2	115.7	117.2	118.7	120.2	121.7	123.2	124.7	126.2
	3/9	83.8	85.0	86.1	87.3	88.4	89.6	90.7	91.8	93.0	94.1	95.2
	4/9	60.2	61.1	61.9	62.7	63.5	64.3	65.0	65.8	66.6	67.4	68.2
	5/9	40.2	40.7	41.2	41.8	42.4	42.9	43.4	44.0	44.6	45.1	45.6
	6/9	23.4	23.8	24.1	24.5	24.8	25.2	25.5	25.9	26.2	26.5	26.8
	7/9	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
10	1/10	157.4	159.6	161.8	164.0	166.2	168.4	170.6	172.8	174.9	177.1	179.3
	2/10	126.8	128.6	130.3	132.1	133.8	135.6	137.3	139.0	140.8	142.5	144.2
	3/10	99.2	100.6	102.0	103.3	104 7	106.1	107.5	108.8	110.2	111.5	112.9
	4/10	75.0	76.0	77.0	78.0	79.0	80.0	81.0	82.0	83.0	84.0	85.0
	5/10	53.9	54.6	55.3	56.1	56.8	57.5	58.2	58.9	59.6	60.4	61.1
	6/10	35.8	36.3	36.8	37.3	37.8	38.3	38.8	39.3	39.7	40.2	40.7
	7/10	20.9	21.2	21.5	21.8	22.1	22.4	22.7	23.0	23.3	23.6	23.9
	8/10	9.6	9.7	9.8	10.0	10.1	10.3	10.4	10.6	10.7	10.9	11.0

TABLE 16.—(*Continued*).

TABLE 16.—(*Continued*).

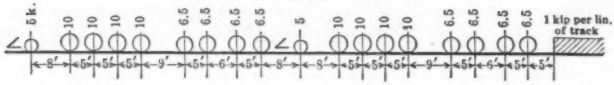
PANEL LENGTHS.

Number of panels.	Panel point.	35′ 6″	36′ 0″	37′ 0″	38′ 0″	39′ 0″	40′ 0″	41′ 0″	42′ 0″	43′ 0″	44′ 0″	45′ 0″
3	1/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/3	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
4	1/4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	3/4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
5	1/5	88.1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	2/5	52.4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	3/5	24.7	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
6	1/6	107.4	108.4	111.0	113.5	115.9	118.2	120.6	123.0	125.5	128.0	130.5
	2/6	71.8	72.6	74.1	75.7	77.3	78.8	80.5	82.1	83.7	85.3	86.9
	3/6	42.6	43.1	44.1	45.1	46.0	47.0	48.0	48.9	49.8	50.8	51.7
	4/6	20.0	20.2	20.7	21.2	21.7	22.2	22.7	23.2	23.7	24.2	24.6
7	1/7	126.3	127.7	130.7	133.6	136.4	139.3	142.2	145.0	147.9	150.9	153.9
	2/7	90.6	91.7	93.7	95.7	97.8	99.9	101.9	104.0	106.0	108.1	110.1
	3/7	60.5	61.2	62.6	64.0	65.4	66.7	68.1	69.4	70.7	72.0	73.3
	4/7	35.9	36.3	37.1	38.0	38.8	39.5	40.3	41.1	41.9	42.7	43.5
	5/7	16.6	16.9	17.3	17.8	18.2	18.6	19.1	19.5	19.9	20.3	20.7
8	1/8	144.9	146.6	150.0	153.4	156.7	160.1	163.4	166.8	170.2	173.6	177.1
	2/8	109.2	110.5	113.0	115.5	118.0	120.6	123.1	125.6	128.1	130.6	133.1
	3/8	78.5	79.4	81.2	83.0	84.7	86.5	88.3	90.0	91.7	93.5	95.2
	4/8	52.4	52.9	54.1	55.2	56.4	57.6	58.8	60.0	61.2	62.3	63.5
	5/8	31.0	31.4	32.1	32.8	33.5	34.2	34.9	35.6	36.3	37.0	37.7
	6/8	14.2	14.4	14.8	15.2	15.6	15.9	16.3	16.7	17.1	17.5	17.8
9	1/9	163.3	165.2	169.1	173.0	176.9	180.7	184.5	188.4	192.2	196.1	200.1
	2/9	127.7	129.2	132.1	135.1	138.1	141.1	144.1	147.1	150.1	153.1	156.0
	3/9	96.3	97.5	99.7	101.9	104.1	106.3	108.5	110.7	112.9	115.0	117.2
	4/9	69.0	69.8	71.4	73.0	74.6	76.2	77.8	79.4	80.9	82.5	84.0
	5/9	46.2	46.7	47.7	48.7	49.8	50.8	51.8	52.8	53.8	54.8	55.8
	6/9	27.1	27.5	28.2	28.9	29.5	30.1	30.8	31.4	32.0	32.6	33.2
	7/9	.....	12.7	13.1	13.4	13.7	14.0	14.3	14.6	14.9	15.3	15.6
10	1/10	181.5	183.7	188.1	192.5	196.9	201.2	205.5	209.8	214.2	218.6	222.9
	2/10	146.0	147.7	151.1	154.6	158.1	161.6	165.1	168.5	172.0	175.4	178.9
	3/10	114.2	115.6	118.2	120.9	123.6	126.2	128.8	131.5	134.2	136.8	139.5
	4/10	86.1	87.1	89.1	91.1	93.1	95.1	97.1	99.1	101.1	103.0	105.0
	5/10	61.8	62.5	63.9	65.3	66.7	68.1	69.5	70.9	72.3	73.7	75.1
	6/10	41.2	41.7	42.6	43.6	44.5	45.4	46.4	47.3	48.2	49.1	50.0
	7/10	24.2	24.4	25.0	25.6	26.3	26.8	27.4	28.0	28.5	29.1	29.6
	8/10	11.2	11.3	11.7	12.0	12.3	12.5	12.8	13.1	13.4	13.7	14.0

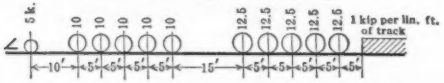
TABLE 17.—Conversion Table, Class E to Class M Engine Loading.

To Convert E-Rating to Equivalent M-Rating Multiply by the Coefficients in this Table
To Convert M-Rating to Equivalent E-Rating Divide by the Coefficients in this Table

Class E-10 Engine Loading



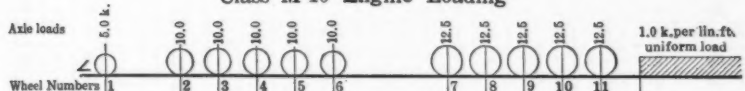
Class M-10 Engine Loading



Span length, in feet	Maximum moment	Maximum shear	Maximum floor beam reaction	Span length, in feet	Maximum moment	Maximum shear	Maximum floor beam reaction
10	0.800	0.800	0.800	46	0.755	0.771	0.777
11	0.801	0.799	0.769	48	0.759	0.771	0.777
12	0.800	0.800	0.747	50	0.765	0.775	0.778
13	0.800	0.800	0.732	52	0.769	0.772	0.782
14	0.801	0.799	0.730	54	0.769	0.767	0.786
15	0.800	0.800	0.729	56	0.774	0.760	0.793
16	0.800	0.800	0.727	58	0.777	0.759	0.803
17	0.801	0.800	0.725	60	0.779	0.756	0.812
18	0.800	0.800	0.724	62	0.781	0.756	0.818
19	0.800	0.799	0.729	64	0.782	0.759	0.825
20	0.800	0.800	0.739	66	0.785	0.760	0.831
21	0.800	0.786	0.746	68	0.785	0.766	0.837
22	0.786	0.773	0.754	70	0.786	0.770	0.845
23	0.772	0.763	0.757	72	0.784	0.775	0.850
24	0.760	0.760	0.761	74	0.781	0.784	0.857
25	0.751	0.758	0.764	76	0.782	0.790	0.862
26	0.743	0.756	0.765	78	0.778	0.795	0.867
27	0.735	0.752	0.770	80	0.776	0.801	0.873
28	0.731	0.749	0.774	82	0.775	0.808	0.877
29	0.729	0.746	0.778	84	0.775	0.815	0.881
30	0.730	0.748	0.778	86	0.775	0.825	0.885
31	0.729	0.750	0.779	88	0.774	0.828	0.887
32	0.728	0.752	0.782	90	0.774	0.839	0.893
33	0.726	0.751	0.786	92	0.774	0.850	0.895
34	0.726	0.752	0.788	94	0.777	0.854	0.898
35	0.724	0.751	0.789	96	0.777	0.855	0.901
36	0.726	0.754	0.786	98	0.777	0.859	0.906
37	0.728	0.753	0.782	100	0.776	0.858	0.909
38	0.731	0.757	0.781	125	0.823	0.874	0.937
39	0.732	0.758	0.782	150	0.859	0.883	0.953
40	0.738	0.759	0.782	175	0.886	0.893	0.964
42	0.745	0.768	0.779	200	0.905	0.902	0.971
44	0.752	0.768	0.776	250	0.939	0.913	0.981

TABLE 18.—MOMENTS, IN FOOT-KIPS, FOR CLASS M-10 ENGINE LOADING.

Class M-10 Engine Loading



Summations	Kips	5.0	15.0	25.0	35.0	45.0	55.0	67.5	80.0	92.5	105.0	117.5	117.5
	Feet	0	10	15	20	25	30	45	50	55	60	65	70
	Kips	117.5	112.5	102.5	92.5	82.5	72.5	62.5	50.0	37.5	25.0	12.5	0
	Feet	70	60	55	50	45	40	25	20	15	10	5	0

Moments of Wheel Loads About Wheel Number												
End of Train		3787.5	3437.5	2837.5	2287.5	1787.5	1337.5	937.5	625.0	375.0	187.5	62.5
	11	3200.0	2875.0	2325.0	1825.0	1375.0	975.0	625.0	375.0	187.5	62.5	
	10	2675.0	2375.0	1875.0	1425.0	1025.0	675.0	375.0	187.5	62.5	62.5	
	9	2212.5	1937.5	1487.5	1087.5	737.5	437.5	187.5	62.5	62.5	187.5	
	8	1812.5	1562.5	1162.5	812.5	512.5	262.5	62.5	62.5	187.5	375.0	
	7	1475.0	1250.0	900.0	600.0	350.0	150.0	62.5	187.5	375.0	625.0	
	6	650.0	500.0	300.0	150.0	50.0	187.5	437.5	750.0	1125.0	1562.5	
	5	425.0	300.0	150.0	50.0	50.0	300.0	612.5	987.5	1425.0	1925.0	
	4	250.0	150.0	50.0	50.0	150.0	462.5	837.5	1275.0	1775.0	2337.5	
	3	125.0	50.0	50.0	150.0	300.0	675.0	1112.5	1612.5	2175.0	2800.0	
	2	50.0	50.0	150.0	300.0	500.0	937.5	1437.5	2000.0	2625.0	3312.5	
	1	100.0	250.0	450.0	700.0	1000.0	1562.5	2187.5	2875.0	3625.0	4437.5	

TABLE 19.—WHEEL DETERMINING MAXIMUM MOMENT FOR CLASS M ENGINE LOADING.

Segments in feet	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	100	110
1000 to 300	2	3	3	3	4	4	5	5	6	6	7	7	8	8	9	9	9	10	10	11
290 to 250	2	3	3	3	4	4	5	5	6	7	7	7	8	8	9	9	9	10	10	11
240	2	3	3	3	4	4	5	6	6	7	7	7	8	8	9	9	9	10	10	11
230	2	3	3	3	4	4	5	6	6	7	7	7	8	8	9	9	9	10	10	11
220	2	3	3	3	4	4	5	6	6	7	7	8	8	8	9	9	9	10	10	11
210	2	3	3	3	4	4	5	6	6	7	7	8	8	8	9	9	9	10	10	11
200 to 160	2	3	3	3	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10	11
150	2	3	3	3	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10	11
140	2	3	3	4	4	5	5	6	6	7	7	8	8	9	9	9	10	10	10	11
130	2	3	3	4	4	5	5	6	7	7	7	8	8	9	9	9	10	10	10	11
120	2	3	3	4	4	5	5	6	7	7	8	8	8	9	9	9	10	10	10	11
110	2	3	4	4	4	5	5	6	7	7	8	8	8	9	9	9	10	10	10	11
100	2	3	4	4	4	5	6	6	7	7	8	8	8	9	9	9	10	10	10	11
95	2	3	4	4	4	5	6	6	7	7	8	8	8	9	9	9	10	10	10	
90	$\overline{11}$	3	4	4	4	5	6	6	7	7	8	8	8	9	9	9	10	10		
85	$\overline{11}$	$\overline{11}$	$\overline{11}$	$\overline{11}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	7	7	7	8	8	9	9	9	9	10			
80	$\overline{11}$	$\overline{11}$	$\overline{11}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	7	8	8	9	9	9	10				
75	$\overline{11}$	$\overline{11}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{9}$	8	8	9	9	9					
70	$\overline{11}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	8	8	9	9						
65	$\overline{11}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	9							
60	$\overline{11}$	$\overline{10}$	$\overline{10}$	$\overline{10}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	8								
55	$\overline{11}$	$\overline{10}$	$\overline{10}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{8}$	$\overline{8}$									
50	7	$\overline{10}$	$\overline{10}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{8}$	$\overline{8}$	$\overline{8}$										
45	7	8	$\overline{10}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{8}$	$\overline{8}$											
40	7	8	8	$\overline{9}$	$\overline{9}$	$\overline{9}$	$\overline{8}$	$\overline{8}$												
35	7	8	8	9	$\overline{9}$	$\overline{9}$	$\overline{8}$													
30	7	8	9	9	9	$\overline{9}$														
25	7	8	9	9	9															
20	7	8	9	9																
15	7	8	9																	
10	7	8																		
5	7																			

The shorter segment is ahead followed by the longer one except where the wheel is overlined

Class M Engine

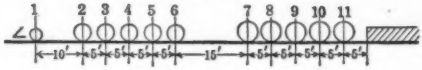
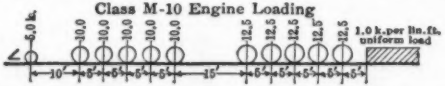


TABLE 20.—MAXIMUM MOMENTS, SHEARS AND REACTIONS, CLASS M-10 ENGINE LOADING, ONE TRACK

TABLE 20.—MAXIMUM MOMENTS, SHEARS AND REACTIONS, CLASS M-10 ENGINE LOADING, ONE TRACK.

Class M-10 Engine Loading

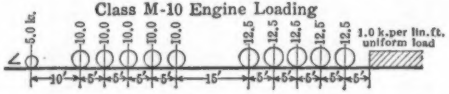


Span, in feet.	Maximum moment, in foot-kips.	Maximum shear, in kips.	Maximum floor-beam reaction, in kips.	Equivalent Uniform Load.			Span, in feet.	Maximum moment, in foot-kips.	Maximum shear, in kips.	Maximum floor-beam reaction, in kips.	Equivalent Uniform Load.		
				Moment.	Shear.	Reaction.					Moment.	Shear.	Reaction.
10	85.2	18.8	25.0	2.81	3.75	2.50	46	549.3	53.7	77.7	2.08	2.33	1.69
11	41.1	20.5	28.4	2.71	3.72	2.58	48	585.4	55.0	80.3	2.03	2.29	1.67
12	50.0	21.9	31.2	2.78	3.65	2.60	50	622.1	56.2	82.8	1.99	2.25	1.66
13	59.4	23.1	33.7	2.81	3.55	2.59	52	660.3	57.8	85.3	1.95	2.22	1.64
14	68.8	24.1	35.7	2.81	3.44	2.55	54	702.3	59.4	87.7	1.93	2.20	1.62
15	78.1	25.0	37.5	2.78	3.33	2.50	56	744.8	61.2	90.0	1.90	2.18	1.61
16	87.5	26.6	39.1	2.73	3.32	2.44	58	788.0	62.9	92.2	1.87	2.17	1.59
17	96.8	27.9	40.6	2.68	3.29	2.39	60	833.6	64.6	94.4	1.85	2.15	1.57
18	106.2	29.2	41.9	2.62	3.24	2.33	62	881.9	66.1	96.6	1.84	2.13	1.56
19	116.6	30.3	43.2	2.58	3.19	2.27	64	930.9	67.6	98.8	1.82	2.11	1.54
20	128.9	31.2	44.4	2.58	3.12	2.22	66	980.4	69.0	100.8	1.80	2.09	1.53
21	141.2	32.7	45.5	2.56	3.12	2.17	68	1 032.7	70.4	102.9	1.79	2.07	1.51
22	156.2	34.1	46.6	2.58	3.10	2.12	70	1 086.3	71.8	104.8	1.77	2.05	1.50
23	171.9	35.3	47.6	2.60	3.07	2.07	72	1 144.2	73.0	106.7	1.77	2.03	1.47
24	187.5	36.5	48.6	2.60	3.04	2.02	74	1 202.8	74.2	108.6	1.76	2.01	1.46
25	203.1	37.5	49.5	2.60	3.00	1.98	76	1 263.0	75.4	110.4	1.75	1.98	1.45
26	218.8	38.5	50.8	2.59	2.96	1.95	78	1 326.6	76.5	112.2	1.74	1.96	1.44
27	234.4	39.4	52.0	2.57	2.92	1.93	80	1 391.0	77.5	113.9	1.74	1.94	1.42
28	250.0	40.3	53.2	2.55	2.88	1.90	82	1 455.7	78.5	115.7	1.73	1.91	1.41
29	265.6	41.2	54.3	2.53	2.84	1.87	84	1 521.5	79.4	117.5	1.72	1.89	1.40
30	281.2	42.1	55.4	2.50	2.81	1.85	86	1 587.8	80.3	119.2	1.72	1.87	1.39
31	296.9	42.9	56.8	2.47	2.77	1.83	88	1 654.7	81.1	121.0	1.71	1.84	1.38
32	312.8	43.7	58.1	2.44	2.73	1.82	90	1 722.2	81.9	122.6	1.70	1.82	1.36
33	328.8	44.5	59.4	2.42	2.70	1.80	92	1 790.3	82.7	124.3	1.69	1.80	1.35
34	344.9	45.3	60.6	2.39	2.66	1.78	94	1 859.2	83.4	125.9	1.68	1.78	1.34
35	361.2	46.1	61.8	2.36	2.63	1.76	96	1 928.5	84.7	127.5	1.67	1.77	1.33
36	377.6	46.8	63.3	2.33	2.60	1.76	98	1 998.5	86.1	129.1	1.66	1.76	1.32
37	394.1	47.7	64.9	2.30	2.58	1.75	100	2 071.8	87.4	130.6	1.66	1.75	1.31
38	410.8	48.3	66.3	2.28	2.54	1.74	125	3 036.3	102.9	149.9	1.56	1.65	1.20
39	427.6	49.0	67.7	2.25	2.51	1.74	150	4 109.1	117.4	170.8	1.46	1.56	1.14
40	444.6	49.7	69.1	2.22	2.48	1.73	175	5 286.1	131.4	192.8	1.38	1.50	1.10
42	478.6	51.1	72.1	2.17	2.43	1.72	200	6 555.2	144.9	215.6	1.31	1.45	1.08
44	513.9	52.4	75.0	2.12	2.38	1.70	250	9 370.3	171.4	262.5	1.20	1.37	1.05

NOTE.—1 k. = 1 000 lb.

TABLE 21.—MAXIMUM MOMENTS, IN FOOT-KIPS, FOR TRUSS BRIDGES, CLASS M-10 ENGINE-LOADING, ONE TRACK.

Class M-10 Engine Loading



PANEL LENGTHS.

Number of panels.	Panel point.	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	12'-6"	13'-0"
3	1/3	167	188	208	229	250	271	292	314	336	358	380
4	1/4	236	262	287	314	341	368	396	425	454	484	514
	2/4	313	345	377	410	444	478	512	547	583	619	660
5	1/5	289	320	351	382	415	448	483	519	561	603	649
	2/5	429	472	516	561	609	654	702	751	809	870	932
6	1/6	335	370	406	447	490	537	584	631	679	729	779
	2/6	527	581	636	692	754	826	899	976	1 056	1 138	1 220
	3/6	583	639	692	766	831	905	980	1 056	1 140	1 230	1 320
7	1/7	377	427	475	523	571	622	673	723	775	826	880
	2/7	620	689	762	845	929	1 013	1 098	1 187	1 277	1 370	1 465
	3/7	732	810	895	985	1 077	1 179	1 290	1 394	1 504	1 618	1 734
8	1/8	436	485	536	588	639	691	747	801	855	910	967
	2/8	728	812	902	993	1 084	1 180	1 277	1 375	1 473	1 572	1 672
	3/8	888	993	1 100	1 217	1 336	1 456	1 578	1 706	1 835	1 965	2 097
	4/8	930	1 080	1 140	1 260	1 381	1 515	1 650	1 788	1 921	2 069	2 217
9	1/9	483	536	592	646	700	755	809	871	938	1 007	1 078
	2/9	838	930	1 027	1 125	1 224	1 323	1 424	1 525	1 627	1 729	1 853
	3/9	1 056	1 178	1 302	1 431	1 563	1 696	1 831	1 969	2 112	2 256	2 402
	4/9	1 187	1 277	1 418	1 562	1 708	1 862	2 018	2 182	2 348	2 517	2 687
10	1/10	530	584	639	694	749	817	886	957	1 030	1 106	1 183
	2/10	955	1 033	1 133	1 233	1 335	1 436	1 545	1 671	1 800	1 933	2 068
	3/10	1 207	1 341	1 476	1 618	1 761	1 858	2 053	2 202	2 351	2 503	2 674
	4/10	1 359	1 516	1 675	1 840	2 010	2 182	2 357	2 534	2 713	2 895	3 101
	5/10	1 381	1 548	1 719	1 892	2 069	2 255	2 444	2 636	2 831	3 036	3 244

TABLE 21.—(*Continued.*)

PANEL LENGTHS.

Number of panels.	Panel point.	13′-6″	14′-0″	14′-6″	15′-0″	15′-6″	16′-0″	16′-6″	17′-0″	17′-6″	18′-0″	18′-6″
3	1/3	404	427	451	475	501	527	554	581	608	636	664
4	1/4	545	576	608	641	683	728	769	812	857	902	947
	2/4	702	745	788	831	880	930	980	1 030	1 081	1 140	1 200
5	1/5	692	741	788	835	884	933	988	1 033	1 083	1 133	1 183
	2/5	996	1 065	1 135	1 205	1 282	1 359	1 437	1 516	1 595	1 675	1 756
6	1/6	829	880	930	981	1 032	1 084	1 137	1 191	1 245	1 306	1 370
	2/6	1 302	1 387	1 475	1 568	1 651	1 741	1 831	1 922	2 017	2 112	2 208
	3/6	1 414	1 515	1 616	1 719	1 822	1 927	2 033	2 143	2 255	2 368	2 482
7	1/7	932	988	1 042	1 098	1 164	1 232	1 300	1 370	1 441	1 514	1 587
	2/7	1 560	1 657	1 754	1 852	1 950	2 050	2 150	2 250	2 352	2 468	2 588
	3/7	1 852	1 973	2 097	2 223	2 350	2 479	2 608	2 739	2 871	3 005	3 155
8	1/8	1 033	1 100	1 170	1 242	1 316	1 392	1 469	1 548	1 628	1 709	1 793
	2/8	1 772	1 873	1 976	2 097	2 220	2 345	2 473	2 602	2 734	2 872	3 013
	3/8	2 223	2 362	2 503	2 645	2 789	2 936	3 099	3 264	3 436	3 611	3 788
	4/8	2 368	2 520	2 675	2 831	2 995	3 160	3 328	3 497	3 669	3 842	4 018
9	1/9	1 151	1 225	1 301	1 379	1 461	1 545	1 631	1 718	1 807	1 898	1 991
	2/9	1 981	2 113	2 247	2 388	2 523	2 665	2 810	2 957	3 108	3 261	3 419
	3/9	2 549	2 698	2 862	3 038	3 222	3 409	3 599	3 791	3 988	4 187	4 394
	4/9	2 860	3 038	3 225	3 415	3 608	3 804	4 008	4 215	4 425	4 638	4 853
10	1/10	1 263	1 345	1 428	1 514	1 604	1 696	1 789	1 885	1 983	2 083	2 185
	2/10	2 210	2 356	2 505	2 658	2 813	2 972	3 134	3 299	3 468	3 639	3 814
	3/10	2 854	3 044	3 241	3 441	3 650	3 863	4 080	4 300	4 524	4 752	4 983
	4/10	3 314	3 530	3 749	3 973	4 199	4 430	4 664	4 905	5 153	5 403	5 658
	5/10	3 455	3 669	3 886	4 106	4 330	4 556	4 792	5 031	5 273	5 519	5 767

TABLE 21.—(*Continued.*)

Number of panels.	Panel point.	Panel Lengths. 19′-0″	19′-6″	20′-0″	20′-6″	21′-0″	21′-6″	22′-0″	22′-6″	23′-0″	23′-6″	24′-0″
3	1/3	692	721	754	791	826	862	899	935	976	1 016	1 056
4	1/4	993	1 039	1 084	1 132	1 180	1 229	1 277	1 326	1 375	1 424	1 473
	2/4	1 260	1 320	1 381	1 448	1 515	1 582	1 650	1 719	1 788	1 857	1 927
5	1/5	1 233	1 284	1 335	1 386	1 437	1 490	1 545	1 608	1 671	1 735	1 800
	2/5	1 840	1 925	2 010	2 096	2 182	2 269	2 357	2 245	2 534	2 623	2 713
6	1/6	1 435	1 502	1 569	1 639	1 710	1 783	1 856	1 931	2 006	2 083	2 160
	2/6	2 305	2 402	2 500	2 599	2 698	2 804	2 920	3 038	3 160	3 284	3 409
	3/6	2 597	2 714	2 831	2 954	3 077	3 202	3 328	3 455	3 533	3 712	3 842
7	1/7	1 662	1 739	1 816	1 897	1 979	2 063	2 148	2 234	2 321	2 410	2 500
	2/7	2 711	2 835	2 961	3 088	3 218	3 356	3 496	3 638	3 786	3 935	4 086
	3/7	3 303	3 604	3 604	3 757	3 911	4 067	4 229	4 393	4 559	4 726	4 896
8	1/8	1 877	1 964	2 052	2 143	2 236	2 331	2 427	2 525	2 624	2 725	2 827
	2/8	3 155	3 300	3 447	3 596	3 748	3 901	4 057	4 216	4 387	4 561	4 738
	3/8	3 967	4 149	4 333	4 519	4 708	4 899	5 092	5 286	5 489	5 694	5 902
	4/8	4 195	4 375	4 556	4 745	4 935	5 128	5 322	5 519	5 717	5 918	6 120
9	1/9	2 085	2 181	2 279	2 381	2 485	2 591	2 698	2 807	2 918	3 031	3 145
	2/9	3 581	3 746	3 914	4 084	4 258	4 434	4 612	4 794	4 978	5 171	5 373
	3/9	4 604	4 817	5 033	5 252	5 474	5 699	5 927	6 158	6 392	6 631	6 876
	4/9	5 071	5 292	5 515	5 742	5 975	6 218	6 458	6 696	6 942	7 190	7 442
10	1/10	2 290	2 396	2 504	2 614	2 728	2 844	2 963	3 083	3 206	3 330	3 457
	2/10	3 994	4 179	4 368	4 559	4 754	4 952	5 153	5 358	5 565	5 776	5 996
	3/10	5 222	5 466	5 714	5 965	6 220	6 478	6 741	7 007	7 276	7 549	7 828
	4/10	5 916	6 177	6 443	6 711	6 984	7 265	7 551	7 840	8 133	8 429	8 729
	5/10	6 019	6 273	6 531	6 798	7 069	7 342	7 619	7 898	8 181	8 461	8 758

TABLE 21.—(*Continued.*)

PANEL LENGTHS.

Number of panels.	Panel point.	24′-6″	25′-0″	25′-6″	26′-0″	26′-6″	27′-0″	27′-6″	28′-0″	28′-6″	29′-0″	29′-6″
3	1/3	1 097	1 138	1 178	1 220	1 261	1 302	1 344	1 387	1 431	1 475	1 518
4	1/4	1 522	1 572	1 622	1 672	1 722	1 772	1 823	1 873	1 924	1 976	2 036
	2/4	1 908	2 069	2 143	2 217	2 292	2 368	2 444	2 520	2 597	2 675	2 753
5	1/5	1 866	1 933	2 000	2 068	2 138	2 210	2 282	2 356	2 430	2 505	2 581
	2/5	2 804	2 895	2 996	3 101	3 207	3 314	3 421	3 530	3 639	3 749	3 860
6	1/6	2 239	2 319	2 399	2 481	2 564	2 648	2 733	2 819	2 908	2 998	3 089
	2/6	3 585	3 668	3 792	3 922	4 054	4 187	4 325	4 464	4 604	4 746	4 889
	3/6	3 974	4 106	4 240	4 375	4 511	4 650	4 792	4 935	5 079	5 225	5 371
7	1/7	2 591	2 684	2 778	2 873	2 969	3 067	3 166	3 266	3 368	3 471	3 576
	2/7	4 238	4 393	4 549	4 707	4 867	5 029	5 192	5 357	5 526	5 698	5 872
	3/7	5 067	5 239	5 414	5 590	5 768	5 947	6 129	6 314	6 503	6 694	6 886
8	1/8	2 931	3 036	3 143	3 251	3 361	3 473	3 586	3 700	3 816	3 934	4 053
	2/8	4 916	5 097	5 280	5 465	5 653	5 842	6 034	6 229	6 425	6 624	6 827
	3/8	6 111	6 323	6 538	6 755	6 974	7 195	7 419	7 645	7 875	8 111	8 349
	4/8	6 325	6 531	6 745	6 960	7 178	7 397	7 619	7 842	8 068	8 295	8 525
9	1/9	3 261	3 379	3 499	3 621	3 744	3 869	3 996	4 125	4 255	4 387	4 520
	2/9	5 577	5 783	5 993	6 205	6 420	6 637	6 858	7 081	7 306	7 535	7 766
	3/9	7 124	7 375	7 629	7 886	8 146	8 409	8 675	8 944	9 216	9 484	9 777
	4/9	7 696	7 953	8 213	8 483	8 754	9 028	9 304	9 588	9 865	10 150	10 433
10	1/10	3 586	3 716	3 849	3 984	4 120	4 259	4 400	4 542	4 687	4 834	4 983
	2/10	6 225	6 458	6 693	6 932	7 174	7 419	7 668	7 919	8 174	8 432	8 693
	3/10	8 114	8 404	8 697	8 994	9 294	9 599	9 907	10 218	10 533	10 852	11 180
	4/10	9 033	9 340	9 652	9 972	10 296	10 624	10 955	11 290	11 628	11 970	12 312
	5/10	9 061	9 370	9 686	10 008	10 336	10 670	11 011	11 358	11 711	12 070	12 436

TABLE 21.—(*Continued*)

PANEL LENGTHS.

Number of panels.	Panel point.	30′-0″	30′-6	31′-0″	31′-6″	32′-0″	32′-6″	33′-0″	33 -6″	34′-0″	34′-6″	35′-0″
3	1/3	1 563	1 607	1 651	1 696	1 741	1 785	1 831	1 876	1 922	1 969	2 017
4	1/4	2 097	2 158	2 220	2 282	2 345	2 409	2 473	2 537	2 602	2 668	2 734
	2/4	2 831	2 913	2 995	3 077	3 160	3 244	3 328	3 412	3 497	3 583	3 669
5	1/5	2 658	2 735	2 813	2 892	2 972	3 053	3 134	3 216	3 299	3 383	3 468
	2/5	3 973	4 085	4 199	4 314	4 430	4 546	4 664	4 783	4 905	5 028	5 153
6	1/6	3 181	3 274	3 369	3 464	3 560	3 659	3 756	3 856	3 956	4 058	4 160
	2/6	5 033	5 179	5 326	5 474	5 624	5 775	5 927	6 081	6 236	6 392	6 550
	3/6	5 519	5 667	5 817	5 968	6 120	6 273	6 428	6 584	6 745	6 906	7 069
7	1/7	3 684	3 793	3 903	4 014	4 127	4 241	4 356	4 473	4 591	4 710	4 830
	2/7	6 048	6 226	6 405	6 587	6 770	6 954	7 141	7 330	7 520	7 712	7 905
	3/7	7 080	7 276	7 474	7 673	7 874	8 077	8 281	8 487	8 697	8 911	9 127
8	1/8	4 173	4 298	4 424	4 551	4 680	4 811	4 943	5 076	5 211	5 348	5 486
	2/8	7 034	7 244	7 455	7 669	7 885	8 103	8 324	8 546	8 771	8 999	9 228
	3/8	8 589	8 832	9 077	9 324	9 573	9 825	10 080	10 336	10 595	10 861	11 130
	4/8	8 758	9 000	9 246	9 496	9 750	10 008	10 270	10 536	10 806	11 080	11 358
9	1/9	4 657	4 794	4 936	5 079	5 224	5 371	5 520	5 669	5 822	5 976	6 132
	2/9	8 001	8 242	8 485	8 731	8 980	9 231	9 485	9 742	10 002	10 264	10 529
	3/9	10 062	10 351	10 644	10 939	11 237	11 538	11 842	12 149	12 457	12 775	13 130
	4/9	10 731	11 033	11 341	11 653	11 971	12 293	12 621	12 953	13 291	13 633	14 206
10	1/10	5 134	5 287	5 442	5 600	5 761	5 925	6 090	6 257	6 427	6 598	6 771
	2/10	8 958	9 227	9 501	9 779	10 061	10 345	10 633	10 923	11 217	11 515	11 815
	3/10	11 511	11 847	12 185	12 528	12 874	13 224	13 578	13 935	14 296	14 663	15 036
	4/10	12 669	13 032	13 401	13 776	14 157	14 544	14 937	15 336	15 741	16 152	16 569
	5/10	12 808	13 186	13 570	13 961	14 358	14 761	15 170	15 586	16 008	16 436	16 870

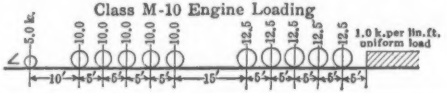
TABLE 21.—(*Continued.*)

PANEL LENGTHS.

Number of panels	Panel point.	35'-6"	36'-0"	37'-0"	38'-0"	39'-0"	40'-0"	41'-0"	42'-0"	43'-0"	44'-0"	45'-0"
3	1/3	2 064	2 112	2 208	2 305	2 402	2 500	2 599	2 698	2 804	2 920	3 038
4	1/4	2 803	2 872	3 013	3 155	3 295	3 447	3 596	3 748	3 901	4 057	4 216
	2/4	3 555	3 842	4 018	4 195	4 370	4 556	4 745	4 935	5 128	5 322	5 519
5	1/5	3 553	3 639	3 814	3 994	4 179	4 368	4 559	4 754	4 952	5 153	5 358
	2/5	5 277	5 403	5 658	5 916	6 177	6 443	6 711	6 984	7 265	7 551	7 840
6	1/6	4 264	4 369	4 581	4 798	5 019	5 248	5 481	5 719	5 960	6 206	6 456
	2/6	6 712	6 876	7 207	7 544	7 886	8 233	8 586	8 944	9 307	9 682	10 063
	3/6	7 232	7 397	7 730	8 068	8 410	8 752	9 122	9 496	9 878	10 270	10 670
7	1/7	4 952	5 075	5 325	5 580	5 840	6 105	6 380	6 660	6 945	7 235	7 530
	2/7	8 101	8 298	8 705	9 120	9 541	9 970	10 405	10 848	11 298	11 757	12 229
	3/7	9 344	9 563	10 006	10 456	10 913	11 380	11 866	12 364	12 874	13 396	13 930
8	1/8	5 626	5 767	6 054	6 347	6 646	6 952	7 264	7 586	7 914	8 249	8 589
	2/8	9 460	9 694	10 171	10 662	11 168	11 672	12 190	12 717	13 254	13 799	14 359
	3/8	11 400	11 673	12 227	12 789	13 361	13 947	14 558	15 177	15 815	16 466	17 135
	4/8	11 640	11 926	12 510	13 110	13 726	14 358	15 006	15 670	16 350	17 046	17 758
9	1/9	6 290	6 449	6 773	7 104	7 442	7 788	8 139	8 503	8 874	9 252	9 638
	2/9	10 797	11 067	11 617	12 183	12 762	13 351	13 952	14 563	15 185	15 819	16 467
	3/9	13 420	13 747	14 410	15 085	15 772	16 477	17 206	17 953	18 718	19 501	20 302
	4/9	14 333	14 691	15 421	16 171	16 941	17 731	18 541	19 371	20 221	21 091	21 904
10	1/10	6 947	7 124	7 485	7 854	8 231	8 616	9 007	9 412	9 826	10 249	10 679
	2/10	12 119	12 425	13 049	13 689	14 345	15 015	15 286	16 393	17 101	17 821	18 558
	3/10	15 413	15 794	16 567	17 355	18 157	18 981	19 831	20 703	21 195	22 509	23 443
	4/10	16 992	17 421	18 297	19 197	20 121	21 069	22 041	23 037	24 057	25 101	26 169
	5/10	17 311	17 758	18 670	19 608	20 570	21 558	22 570	23 608	24 670	25 758	26 870

TABLE 22.—MAXIMUM SHEARS, IN KIPS, FOR TRUSS BRIDGES, CLASS M-10 ENGINE LOADING, ONE TRACK.

Class M-10 Engine Loading



PANEL LENGTHS.

Number of panels.	Panel point.	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	12'-6"	13'-0"
3	1/3	20.8	22.0	23.1	24.1	25.0	25.8	26.6	27.3	28.0	28.6	29.3
	2/3	5.7	5.9	6.0	6.1	6.2	6.7	7.2	7.6	8.0	8.3	8.6
4	1/4	29.5	30.7	31.9	33.0	34.1	35.1	36.0	36.9	37.8	38.7	39.6
	2/4	13.7	14.7	15.6	16.4	17.2	17.9	18.5	19.0	19.5	20.0	20.4
	3/4	4.3	4.4	4.5	4.6	4.7	4.9	5.1	5.3	5.5	5.6	5.8
5	1/5	36.2	37.6	38.9	40.2	41.5	42.7	43.9	45.1	46.7	48.3	49.9
	2/5	21.9	22.7	23.5	24.3	25.1	25.9	26.7	27.5	28.3	29.1	30.0
	3/5	10.6	11.2	11.7	12.1	12.5	13.1	13.6	14.1	14.6	15.0	15.4
6	1/6	41.9	43.6	45.1	47.0	49.0	51.0	53.1	54.9	56.6	58.2	59.8
	2/6	28.6	29.4	30.3	31.5	32.7	34.0	35.3	36.6	37.9	39.2	40.5
	3/6	18.2	19.0	19.7	20.3	20.8	21.3	21.8	22.2	23.7	23.4	24.1
	4/6	8.9	9.3	9.7	10.1	10.4	10.9	11.4	11.8	12.1	12.5	12.8
7	1/7	47.2	50.2	52.8	55.0	57.1	59.1	61.1	62.9	64.6	66.1	67.7
	2/7	34.4	35.8	37.4	39.3	41.1	42.9	44.6	46.1	47.6	49.0	50.4
	3/7	24.5	25.4	26.3	27.2	28.2	29.1	30.0	30.9	31.8	32.8	33.8
	4/7	15.6	16.3	16.9	17.4	17.9	18.4	18.8	19.3	19.7	20.2	20.7
	5/7	7.6	8.0	8.3	8.6	8.9	9.3	9.7	10.1	10.4	10.7	11.0
8	1/8	54.5	57.0	59.5	61.8	63.8	65.8	67.8	69.6	71.3	72.8	74.3
	2/8	41.1	43.0	45.0	46.9	48.8	50.8	52.7	54.1	55.5	56.9	58.2
	3/8	30.1	31.5	32.8	34.2	35.6	36.9	38.3	39.5	40.6	41.8	42.9
	4/8	21.5	22.1	22.7	23.4	24.1	24.9	25.6	26.4	27.2	28.1	29.1
	5/8	13.7	14.2	14.7	15.2	15.6	16.0	16.3	16.6	17.0	17.4	17.8
	6/8	6.6	7.0	7.3	7.6	7.8	8.2	8.5	8.8	9.1	9.4	9.6
9	1/9	60.3	62.9	65.7	67.9	70.0	71.8	73.6	75.6	78.1	80.5	82.9
	2/9	47.8	50.1	52.5	54.5	56.4	58.0	59.6	61.2	62.9	64.6	66.3
	3/9	36.4	37.9	39.6	41.4	43.1	44.8	46.4	47.9	49.3	50.5	51.6
	4/9	26.7	27.8	28.9	30.0	31.1	32.3	33.5	34.7	35.9	37.0	38.1
	5/9	19.1	19.6	20.2	20.8	21.4	22.1	22.8	23.5	24.2	24.9	25.7
	6/9	12.1	12.6	13.1	13.5	13.9	14.2	14.5	14.8	15.1	15.5	15.8
	7/9	5.9	6.2	6.5	6.7	7.0	7.3	7.6	7.9	8.1	8.3	8.5
10	1/10	66.2	68.6	71.0	73.0	74.9	77.7	80.5	83.2	85.9	88.5	91.0
	2/10	54.3	56.6	59.0	60.7	62.5	64.2	66.2	68.1	70.1	72.0	74.0
	3/10	42.8	44.5	46.1	47.8	49.4	51.1	52.7	54.4	56.0	57.5	59.0
	4/10	32.7	34.2	35.7	37.2	38.7	40.1	41.4	42.5	43.6	44.7	45.8
	5/10	24.0	25.0	26.0	26.9	27.8	28.7	29.6	30.5	31.4	32.3	33.2
	6/10	17.2	17.7	18.2	18.7	19.3	19.9	20.5	21.1	21.6	22.2	22.8
	7/10	10.9	11.4	11.8	12.2	12.5	12.8	13.1	13.3	13.6	13.9	14.2
	8/10	5.3	5.6	5.8	6.1	6.3	6.6	6.8	7.1	7.2	7.5	7.7

TABLE 22.—(*Continued.*)

TABLE 22.—(*Continued.*)

PANEL LENGTHS.

Number of panels.	Panel point.	13'-6"	14'-0"	14'-6"	15'-0"	15'-6"	16'-0"	16'-6"	17'-0"	17'-6"	18'-0".	18'-6"
3	1/3	30.0	30.6	31.1	31.7	32.4	33.0	33.6	34.2	34.8	35.4	35.9
	2/3	8.9	9.2	9.5	9.7	10.1	10.4	10.7	11.0	11.3	11.6	11.8
4	1/4	40.4	41.2	42.1	43.3	44.4	45.5	46.5	47.8	49.0	50.1	51.2
	2/4	20.8	21.2	21.5	21.8	22.2	22.7	23.2	23.6	24.0	24.5	25.0
	3/4	5.9	6.0	6.1	6.3	6.5	6.8	7.1	7.3	7.6	7.8	8.0
5	1/5	51.4	53.0	54.5	55.7	57.0	58.3	59.5	60.8	61.9	63.0	64.0
	2/5	30.9	31.7	32.5	33.4	34.3	35.1	35.9	36.7	37.5	38.4	39.2
	3/5	15.7	16.1	16.4	16.7	17.1	17.4	17.8	18.1	18.4	18.8	19.2
6	1/6	61.4	62.8	64.1	65.4	66.7	67.9	69.0	70.1	71.2	72.5	74.0
	2/6	41.8	43.0	44.2	45.3	46.3	47.3	48.3	49.3	50.2	51.0	51.8
	3/6	24.9	25.6	26.3	26.9	27.7	28.5	29.2	29.9	30.6	31.2	31.9
	4/6	13.1	13.4	13.7	13.9	14.1	14.3	14.5	14.7	14.9	15.1	15.4
7	1/7	69.1	70.5	71.9	73.2	75.1	77.0	78.8	80.6	82.4	84.1	85.7
	2/7	51.7	52.9	54.2	55.4	56.6	57.7	58.9	60.0	61.0	62.2	63.3
	3/7	34.9	35.9	36.9	38.9	39.0	40.0	40.8	41.6	42.3	43.0	43.7
	4/7	21.1	21.6	22.0	22.5	23.1	23.7	24.3	24.9	25.5	26.1	26.7
	5/7	11.2	11.5	11.7	11.9	12.1	12.3	12.5	12.6	12.8	13.0	13.2
8	1/8	76.4	78.5	80.7	82.8	84.9	87.0	89.0	91.0	93.0	94.9	96.8
	2/8	59.6	61.0	62.4	63.9	65.3	66.7	68.1	69.6	71.0	72.5	74.0
	3/8	44.1	45.3	46.5	47.4	48.3	49.3	50.2	51.1	52.0	53.1	54.1
	4/8	30.1	31.1	32.0	32.8	33.6	34.4	35.2	35.9	36.5	37.1	37.7
	5/8	18.2	18.6	19.1	19.5	20.0	20.4	20.9	21.5	22.1	22.7	23.2
	6/8	9.8	10.0	10.2	10.4	10.6	10.7	10.9	11.0	11.2	11.4	11.6
9	1/9	85.2	87.5	89.7	91.9	94.2	96.5	98.9	101.0	103.2	105.4	107.7
	2/9	68.0	69.7	71.4	73.1	74.8	76.5	78.2	80.0	81.8	83.5	85.8
	3/9	52.8	53.9	55.1	56.2	57.4	58.5	59.7	60.8	62.2	63.5	64.9
	4/9	39.3	40.3	41.3	42.1	42.9	43.7	44.4	45.1	45.8	46.7	47.6
	5/9	26.4	27.2	28.0	28.7	29.4	30.2	30.9	31.5	32.2	32.8	33.4
	6/9	16.2	16.6	17.0	17.4	17.8	18.2	18.6	19.0	19.4	19.8	20.3
	7/9	8.7	8.9	9.1	9.3	9.4	9.5	9.7	9.8	9.9	10.1	10.2
10	1/10	93.5	96.0	98.4	100.9	103.5	106.0	108.4	110.9	113.3	115.8	118.2
	2/10	76.0	77.9	79.9	81.9	84.0	86.0	88.0	90.1	92.1	94.2	96.2
	3/10	60.5	61.9	63.4	64.9	66.5	68.1	69.6	71.2	72.8	74.4	75.9
	4/10	46.9	48.0	49.1	50.3	51.4	52.5	53.6	54.8	55.9	57.0	58.1
	5/10	34.1	35.0	35.9	36.8	37.7	38.6	39.5	40.4	41.1	41.8	42.5
	6/10	23.4	23.9	24.5	25.1	25.7	26.2	26.8	27.4	27.9	28.5	29.1
	7/10	14.5	14.9	15.3	15.6	16.0	16.4	16.7	17.1	17.5	17.9	18.2
	8/10	7.9	8.0	8.2	8.3	8.5	8.6	8.7	8.8	8.9	9.1	9.2

TABLE 22.—(*Continued.*)

Number of panels.	Panel point.	Panel Lengths.										
		19′-0″	19′-6″	20′-0″	20′-6″	21′-0″	21′-6″	22′-0″	22′-6″	23′-0″	23′-6″	24′-0″
3	1/3	36.4	37.0	37.7	38.5	39.3	40.0	40.8	41.5	42.4	43.2	44.0
	2/3	12.1	12.3	12.5	12.7	12.9	13.1	13.3	13.4	13.6	13.8	14.0
4	1/4	52.2	53.3	54.2	55.2	56.2	57.1	58.0	58.9	59.8	60.6	61.4
	2/4	25.4	25.8	26.3	26.8	27.3	27.8	28.3	28.8	29.3	29.8	30.3
	3/4	8.2	8.4	8.6	8.8	8.9	9.1	9.2	9.4	9.5	9.6	9.8
5	1/5	65.0	65.9	66.8	67.6	68.4	69.3	70.2	71.4	72.7	73.8	75.0
	2/5	40.1	40.9	41.8	42.5	43.2	43.9	44.7	45.4	46.0	46.6	47.2
	3/5	19.6	19.9	20.3	20.7	21.0	21.4	21.8	22.1	22.6	23.0	23.4
6	1/6	75.5	77.0	78.5	80.0	81.4	82.9	84.3	85.8	87.2	88.5	90.0
	2/6	52.6	53.4	54.2	54.9	55.7	56.5	57.3	58.0	59.0	59.9	60.8
	3/6	32.6	33.3	33.9	34.5	35.1	35.7	36.3	36.9	37.4	37.8	38.3
	4/6	15.8	16.1	16.3	16.7	17.0	17.3	17.7	18.0	18.3	18.6	19.0
7	1/7	87.4	89.0	90.7	92.5	94.2	96.0	97.7	99.3	100.9	102.5	104.1
	2/7	64.4	65.5	66.5	67.5	68.6	69.6	70.6	71.9	73.2	74.4	75.6
	3/7	44.4	45.1	45.8	46.4	47.0	47.6	48.2	48.6	49.6	50.4	51.3
	4/7	27.3	27.9	28.6	29.1	29.6	30.1	30.6	31.1	31.5	31.9	32.3
	5/7	13.4	13.7	13.9	14.1	14.3	14.6	14.8	15.0	15.3	15.6	15.9
8	1/8	98.7	100.7	102.6	104.5	106.3	108.2	110.2	112.2	114.1	116.0	117.8
	2/8	75.5	76.9	78.4	79.9	81.4	82.8	84.2	85.7	87.1	88.5	90.0
	3/8	55.1	56.2	57.2	58.2	59.2	60.2	61.3	62.3	63.4	64.4	65.5
	4/8	38.3	38.9	39.5	40.1	40.7	41.3	41.9	42.5	43.1	43.7	44.3
	5/8	23.7	24.2	24.6	25.0	25.5	26.0	26.4	26.8	27.2	27.6	28.0
	6/8	11.8	12.0	12.2	12.4	12.5	12.7	12.9	13.1	13.3	13.5	13.7
9	1/9	109.9	112.0	114.0	116.2	118.2	120.4	122.7	124.8	126.8	128.9	131.1
	2/9	86.9	88.6	90.3	92.0	93.7	95.3	96.9	98.6	100.2	101.8	103.5
	3/9	66.3	67.6	69.0	70.2	71.5	72.8	74.0	75.3	76.6	77.9	79.1
	4/9	48.5	49.4	50.3	51.2	52.2	53.1	54.0	54.9	55.8	56.7	57.6
	5/9	34.0	34.5	35.0	35.5	35.9	36.4	36.8	37.4	38.0	38.6	39.1
	6/9	20.7	21.1	21.5	21.9	22.3	22.7	23.1	23 5	23.9	24.3	24.6
	7/9	10.4	10.6	10.7	10.9	11.1	11.2	11.4	11.6	11.7	11.9	12.1
10	1/10	120.6	122.9	125.2	127.5	130.0	132.3	134.7	137.0	139.4	141.8	144.1
	2/10	98.1	100.0	101.8	103.7	105.7	107.6	109.4	111.3	113.1	115.0	116.8
	3/10	77.5	79.1	80.7	82.2	83.7	85.2	86.7	88.2	89.6	91.1	92.5
	4/10	59.3	60.4	61.5	62.7	63.8	64.9	66.1	67.2	68.3	69.5	70.7
	5/10	43.2	43.8	44.5	45.2	46.0	46.8	47.7	48.5	49.3	50.2	51.0
	6/10	29.7	30.2	30.8	31.4	31.9	32.5	33.1	33.6	34.1	34.6	35.0
	7/10	18.6	19.0	19.3	19.6	20.0	20.4	20.7	21.1	21.4	21.7	22.1
	8/10	9.4	9.5	9.6	9.8	10.0	10.1	10.3	10.4	10.6	10.8	10.9

TABLE 22.—(*Continued.*)

PANEL LENGTHS.

Number of panels.	Panel point.	24'-6"	25'-0"	25'-6"	26'-0"	26'-6"	27'-0"	27'-6"	28'-0"	28'-6"	29'-0"	29'-6"
3	1/3	44.7	45.5	46.2	46.9	47.6	48.3	48.9	49.5	50.2	50.9	51.5
	2/3	14.1	14.3	14.4	14.5	14.7	14.8	14.9	15.1	15.2	15.3	15.5
4	1/4	62.2	62.9	63.6	64.3	65.0	65.6	66.2	66.9	67.5	68.1	69.0
	2/4	30.8	31.3	31.8	32.3	32.8	33.4	33.9	34.4	34.8	35.2	35.7
	3/4	9.9	10.0	10.1	10.2	10.3	10.4	10.5	10.6	10.7	10.8	10.8
5	1/5	76.1	77.3	78.4	79.6	80.7	81.9	83.0	84.1	85.3	86.4	87.5
	2/5	47.8	48.4	48.9	49.5	50.0	50.4	50.9	51.4	52.0	52.5	53.0
	3/5	23.7	24.1	24.5	24.9	25.3	25.7	26.1	26.5	26.8	27.2	27.5
6	1/6	91.4	92.8	94.2	95.5	96.7	98.0	99.3	100.5	102.0	103.4	104.8
	2/6	61.7	62.7	63.7	64.7	65.6	66.5	67.5	68.4	69.3	70.2	71.1
	3/6	38.8	39.2	39.6	40.1	40.6	41.0	41.4	41.9	42.3	42.7	43.1
	4/6	19.3	19.6	19.9	20.3	20.6	20.9	21.2	21.5	21.8	22.1	22.4
7	1/7	105.7	107.3	108.9	110.4	112.0	113.7	115.2	116.7	118.2	119.7	121.2
	2/7	76.8	78.0	79.2	80.4	81.6	82.7	83.8	85.0	86.2	87.3	88.4
	3/7	52.1	52.9	53.7	54.5	55.3	56.1	56.8	57.6	58.4	59.1	59.8
	4/7	32.7	33.0	33.4	33.8	34.2	34.5	34.9	35.3	35.6	36.0	36.3
	5/7	16.2	16.4	16.7	17.0	17.3	17.6	18.0	18.4	18.9	19.3	19.7
8	1/8	119.5	121.4	123.2	125.0	126.8	128.6	130.4	132.2	133.9	135.6	137.3
	2/8	91.4	92.8	94.1	95.5	96.9	98.3	99.6	101.0	102.3	103.6	104.9
	3/8	66.5	67.5	68.4	69.4	70.3	71.3	72.2	73.2	74.2	75.2	76.2
	4/8	44.9	45.6	46.3	47.0	47.7	48.4	49.1	49.8	50.4	51.0	51.6
	5/8	28.2	28.5	28.9	29.2	29.5	29.8	30.2	30.5	30.8	31.2	31.5
	6/8	13.9	14.1	14.4	14.6	14.9	15.1	15.4	15.6	15.8	16.0	16.2
9	1/9	133.1	135.2	137.2	139.2	141.2	143.2	145.2	147.3	149.3	151.2	153.2
	2/9	105.2	106.8	108.5	110.1	111.7	113.3	114.9	116.4	118.0	119.6	121.2
	3/9	80.3	81.5	82.7	83.8	85.0	86.3	87.5	88.7	89.9	91.2	92.4
	4/9	58.4	59.2	60.1	61.0	61.9	62.8	63.7	64.6	65.4	66.2	69.1
	5/9	39.7	40.3	40.9	41.4	42.0	42.6	43.2	43.7	44.3	44.9	45.5
	6/9	24.9	25.2	25.5	25.8	26.1	26.4	26.7	27.0	27.3	27.6	27.9
	7/9	12.3	12.5	12.6	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.2
10	1/10	146.3	148.6	151.0	153.3	155.5	157.8	160.1	162.2	164.4	166.6	168.9
	2/10	118.8	120.6	122.5	124.4	126.2	128.0	129.8	131.6	133.4	135.2	137.0
	3/10	93.9	95.3	96.7	98.1	99.5	101.0	102.5	103.9	105.3	106.7	108.2
	4/10	71.7	72.8	73.9	75.0	76.1	77.1	78.2	79.3	80.3	81.3	82.4
	5/10	51.8	52.7	53.5	54.3	55.1	56.0	56.8	57.6	58.4	59.2	60.0
	6/10	35.5	35.9	36.4	36.9	37.4	37.9	38.3	38.8	39.3	39.8	40.4
	7/10	22.4	22.7	22.9	23.2	23.5	23.7	24.0	24.2	24.4	24.6	24.8
	8/10	11.1	11.3	11.4	11.6	11.8	11.9	12.1	12.3	12.5	12.6	12.8

TABLE 22.—*(Continued.)*

PANEL LENGTHS.

Number of panels.	Panel point.	30′-0″	30′-6″	31′-0″	31′-6″	32′-0″	32′-6″	33′-0″	33′-6″	34′-0″	34′-6″	35′-0″
3	1/3	52.1	52.7	53.3	53.8	54.4	54.9	55.5	56.0	56.6	57.1	57.6
	2/3	15.6	15.8	15.9	16.0	16.2	16.4	16.5	16.6	16.8	17.0	17.1
4	1/4	69.9	70.7	71.5	72.4	73.3	74.1	74.9	75.7	76.5	77.3	78.1
	2/4	36.2	36.6	37.0	37.5	37.9	38.3	38.7	39.1	39.5	39.9	40.2
	3/4	10.9	11.0	11.1	11.2	11.4	11.5	11.6	11.7	11.8	11.9	12.0
5	1/5	88.6	89.6	90.7	91.8	92.8	93.8	94.8	95.9	97.0	98.0	99.0
	2/5	53.6	54.3	54.9	55.5	56.2	56.9	57.5	58.1	58.8	59.4	60.0
	3/5	27.8	28.2	28.5	28.8	29.2	29.5	29.8	30.1	30.4	30.7	31.0
6	1/6	106.0	107.3	108.7	110.0	111.2	112.3	113.6	115.0	116.2	117.6	119.0
	2/6	72.0	72.8	73.6	74.5	75.5	76.4	77.2	78.0	78.8	79.6	80.5
	3/6	43.5	44.0	44.6	45.1	45.7	46.2	46.7	47.2	47.8	48.3	48.8
	4/6	22.6	22.9	23.2	23.5	23.8	24.0	24.3	24.5	24.7	24.9	25.1
7	1/7	123.8	124.3	126.8	127.2	128.6	130.3	132.0	133.7	135.1	136.6	138.0
	2/7	89.4	90.5	91.6	92.7	93.8	94.9	95.9	97.0	98.0	99.1	100.2
	3/7	60.5	61.3	62.1	62.9	63.5	64.2	65.0	65.8	66.5	67.3	68.0
	4/7	36.6	37.1	37.6	38.0	38.5	39.0	39.4	39.8	40.3	40.8	41.2
	5/7	19.1	19.4	19.6	19.8	20.0	20.2	20.4	20.6	20.8	21.0	21.2
8	1/8	139.1	141.0	142.9	144.6	146.3	148.0	149.7	151.5	153.2	155.0	156.8
	2/8	106.3	107.6	108.9	110.2	111.5	112.9	114.2	115.4	116.7	118.0	119.2
	3/8	77.2	78.2	79.1	80.1	81.0	82.0	83.0	83.9	84.8	85.7	86.7
	4/8	52.2	52.8	53.4	54.0	54.6	55.3	56.0	56.7	57.3	58.0	58.6
	5/8	31.8	32.1	32.5	32.8	33.2	33.6	34.0	34.4	34.8	35.2	35.6
	6/8	16.4	16.6	16.8	17.0	17.2	17.4	17.6	17.8	18.0	18.1	18.3
9	1/9	155.2	157.2	159.2	161.2	163.2	165.2	167.3	169.4	171.3	173.2	175.2
	2/9	122.7	124.2	125.7	127.3	128.8	130.3	131.8	133.3	134.8	136.4	137.9
	3/9	93.5	94.7	95.9	97.1	98.3	99.4	100.6	101.7	102.8	104.0	105.2
	4/9	67.9	68.8	69.6	70.4	71.2	72.1	72.9	73.8	74.6	75.4	76.2
	5/9	46.0	46.5	47.1	47.7	48.2	48.8	49.4	49.9	50.5	51.0	51.6
	6/9	28.2	28.5	28.7	29.0	29.3	29.6	29.9	30.2	30.4	30.8	31.1
	7/9	14.4	14.6	14.7	14.9	15.1	15.3	15.5	15.7	15.8	16.0	16.2
10	1/10	171.1	173.3	175.5	177.8	180.0	182.2	184.5	186.8	189.0	191.3	193.5
	2/10	138.8	140.5	142.3	144.1	145.9	147.7	149.4	151.2	153.0	154.7	156.4
	3/10	109.6	111.0	112.4	113.8	115.2	116.6	118.0	119.4	120.7	122.0	123.4
	4/10	83.4	84.4	85.5	86.5	87.6	88.7	89.8	90.8	91.9	93.0	94.0
	5/10	60.7	61.5	62.2	62.9	63.6	64.4	65.2	65.9	66.6	67.4	68.1
	6/10	40.9	41.4	42.0	42.5	43.0	43.5	44.0	44.5	45.1	45.6	46.1
	7/10	25.0	25.3	25.6	25.8	26.1	26.4	26.6	26.9	27.2	27.5	27.7
	8/10	13.0	13.1	13.3	13.4	13.6	13.7	13.9	14.0	14.1	14.3	14.4

TABLE 22.—(*Continued.*)

Number of panels.	Panel point.	Panel Lengths. 35'-6"	36'-0"	37'-0"	38'-0"	39'-0"	40'-0"	41'-0"	42'-0"	43'-0"	44'-0"	45'-0"
3	1/3	58.1	58.6	59.7	60.7	61.6	62.5	63.4	64.3	65.2	66.3	67.5
	2/3	17.2	17.4	17.8	18.1	18.5	18.9	19.3	19.7	20.0	20.4	20.8
4	1/4	79.0	79.8	81.4	83.0	84.6	86.2	87.8	89.3	90.8	92.3	93.7
	2/4	40.5	40.9	41.5	42.1	42.8	43.4	44.0	44.6	45.2	45.8	46.6
	3/4	12.1	12.2	12.5	12.7	12.9	13.2	13.4	13.6	13.9	14.2	14.4
5	1/5	100.1	101.1	103.0	105.0	107.1	109.2	111.2	113.2	115.2	117.1	119.1
	2/5	60.6	61.3	62.6	63.9	65.2	66.4	67.6	68.8	70.0	71.2	72.4
	3/5	31.2	31.5	32.0	32.5	33.0	33.4	33.8	34.2	34.6	35.1	35.7
6	1/6	120.1	121.2	123.7	126.2	128.7	131.2	133.7	136.2	138.7	141.2	143.6
	2/6	81.4	82.3	84.0	85.6	87.3	89.0	90.7	92.3	93.9	95.5	97.1
	3/6	49.3	49.9	51.0	52.0	53.0	54.0	54.9	55.9	56.9	57.9	58.8
	4/6	25.3	25.5	25.9	26.3	26.7	27.1	27.5	27.9	28.3	28.7	29.0
7	1/7	139.5	141.0	144.0	147.0	150.0	153.0	156.0	158.9	161.7	164.5	167.3
	2/7	101.3	102.3	104.5	106.6	108.7	110.8	112.9	115.0	117.0	119.1	121.1
	3/7	68.7	69.4	70.8	72.2	73.6	75.0	76.4	77.7	79.0	80.4	81.8
	4/7	41.6	42.0	42.9	43.8	44.6	45.4	46.2	47.1	47.9	48.8	49.6
	5/7	21.4	21.5	21.8	22.2	22.5	22.9	23.2	23.5	23.8	24.1	24.4
8	1/8	158.5	160.2	163.6	167.0	170.4	173.8	177.2	180.7	184.1	187.6	191.0
	2/8	120.5	121.8	124.5	127.1	129.7	132.2	134.7	137.3	139.8	142.3	144.7
	3/8	87.7	88.6	90.4	92.2	94.0	95.8	97.6	99.4	101.2	103.0	104.7
	4/8	59.2	59.9	61.0	62.2	63.5	64.8	66.1	67.3	68.4	69.6	70.8
	5/8	36.0	36.3	37.1	37.8	38.6	39.3	40.0	40.7	41.4	42.1	42.8
	6/8	18.5	18.7	18.9	19.2	19.5	19.8	20.1	20.4	20.7	20.9	21.2
9	1/9	177.1	179.0	183.0	187.0	190.9	194.7	198.2	202.4	206.4	210.3	214.2
	2/9	139.5	141.0	144.1	147.1	150.2	153.2	156.2	159.3	162.3	165.2	168.0
	3/9	106.3	107.5	109.7	111.8	114.1	116.4	118.6	120.9	123.1	125.4	127.7
	4/9	77.1	77.9	79.6	81.2	82.8	84.4	86.0	87.6	89.2	90.8	92.4
	5/9	52.2	52.7	53.8	54.8	55.9	57.0	58.1	59.2	60.2	61.3	62.3
	6/9	31.5	31.8	32.5	33.2	33.9	34.5	35.1	35.8	36.4	37.0	37.7
	7/9	16.4	16.5	16.8	17.0	17.3	17.5	17.8	18.0	18.2	18.4	18.6
10	1/10	195.7	198.0	202.3	206.7	211.1	215.4	219.7	224.1	228.5	232.9	237.3
	2/10	158.2	160.0	163.5	167.0	170.5	174.0	177.5	181.0	184.4	187.8	191.3
	3/10	124.8	126.2	129.0	131.6	134.3	136.9	139.6	142.3	145.1	147.7	150.3
	4/10	95.0	96.0	98.0	100.1	102.1	104.1	106.1	108.1	110.1	112.0	114.1
	5/10	68.8	69.5	70.9	72.4	73.9	75.4	76.9	78.3	79.7	81.1	82.5
	6/10	46.5	47.1	48.1	49.0	50.0	51.0	51.9	52.8	53.8	54.7	55.6
	7/10	28.0	28.3	28.9	29.5	30.1	30.7	31.3	31.6	32.5	33.0	33.6
	8/10	14.6	14.7	14.9	15.1	15.4	15.6	15.8	16.0	16.2	16.4	16.6

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Paper No. 1519

Discussion

TENTATIVE SPECIFICATIONS FOR STEEL RAILWAY BRIDGE SUPERSTRUCTURE*

By Messrs. F. E. Turneaure, Burton R. Leffler, Francis P. Witmer, James E. Howard, Crosby Miller, C. D. Purdon, G. B. Woodruff, C. E. Conover, Edward Godfrey, D. B. Steinman, Percival S. Baker, A. W. Carpenter, Benjamin W. Guppy, G. G. Thomas, Charles Evan Fowler, Frederick E. Schall, O. H. Ammann, J. B. Hunley, Otis E. Hovey, J. W. Spiller, J. B. French, Henry Goldmark, R. S. Chew, H. M. MacKay, Alfred S. Niles, Jr., and Henry B. Seaman.

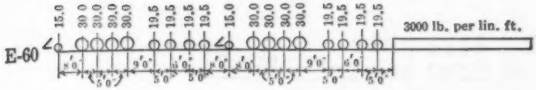
F. E. Turneaure,† M. Am. Soc. C. E. (by letter).—As both live loading and impact have been recent subjects discussed by the American Railway Engineering Association, the work of the Special Committee on Specifications for Bridge Design and Construction of the Society, was greatly simplified, and, in general, consisted of a careful review of the publications of that Association and the consideration of new material or new aspects that may have appeared since that time.

Engine Loading.—A detailed study of the relative effects of various engine loadings in use and the standard Cooper series has been published by the American Railway Engineering Association.‡ The most significant of these diagrams are re-published as Figs. 1 to 6, inclusive, and show clearly the relative effects of the Cooper series as compared with engine loadings in use. Of these diagrams, Fig. 1, Group 1, shows seven heavy road locomotives of various types; Fig. 2, Group 2, shows six locomotives, representing the

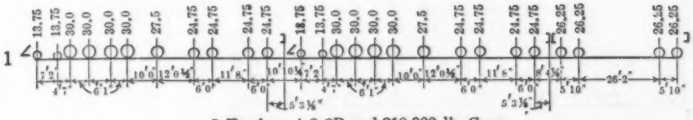
* Abstracts of discussions on the Tentative Specifications for Steel Railway Bridges presented as a Progress Report of the Special Committee on Specifications for Bridge Design and Construction to the Annual Meeting of the Society, and published in *Proceedings*, for December, 1921, April, May, August, and October, 1922. These abstracts are published in *Transactions* by the authority of the Committee on Technical Activities and Publications as valuable contributions to the subject.

† Cons. Engr.; Dean, Coll. of Mechanics and Eng., Univ. of Wisconsin, Madison, Wis.

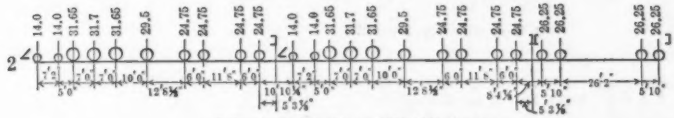
‡ *Proceedings*, Am. Ry. Eng. Assoc., Vol. 21, p. 561.



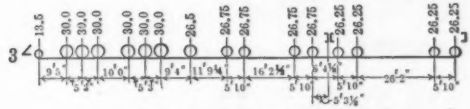
Cooper's E-60



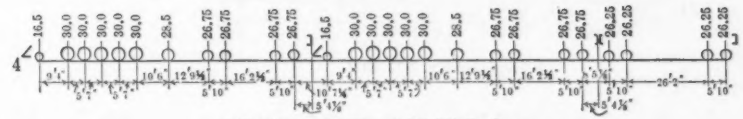
2 Engines 4-8-2B and 210,000-lb. Cars



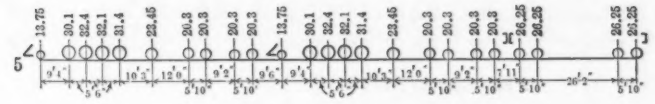
2 Engines 4-6-2B and 210,000-lb. Cars



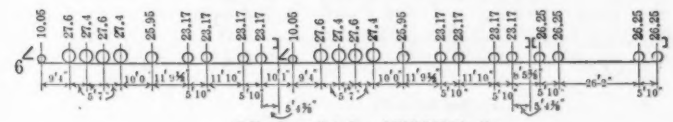
1 Engine 2-6-6-2B and 210,000-lb. Cars



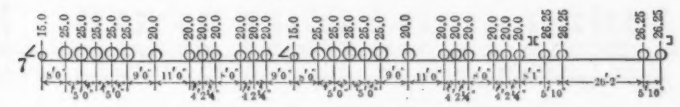
2 Engines 2-10-2B and 210,000-lb. Cars



2 N. Y. C. R. R. Engines H-7A 2-8-2 and 210,000-lb. Cars



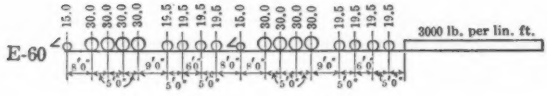
2 Engines 2-8-2 and 210,000-lb. Cars



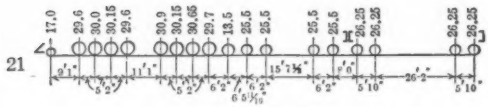
2 Sante Fe Type Engines 2-10-2 and 210,000-lb. Cars

Note: Engines 1 to 7, inclusive, are followed by trains of 210,000-lb. cars. Concentrations are for 1 rail, in thousands of pounds.

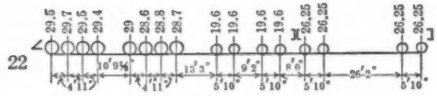
FIG. 1.



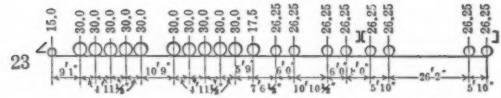
Cooper's E-60



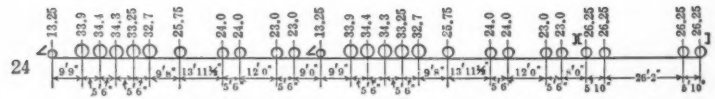
1 Virginian Ry. 2-8-8-2



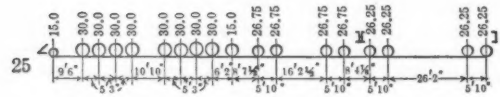
1 L.S. & M.S. Mallet 0-8-8-0



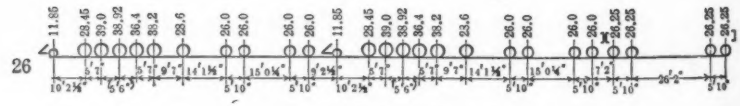
1 Virginian Ry. 2-10-10-2



2 Erie Engines 2-10-2



1 Engine 2-8-8-2B



2 Pennsylvania R. R. NIS Engines 2-10-2

Note: Engines 21 to 26, inclusive, are followed by trains of 210,000-lb. cars. Concentrations are for 1 rail, in thousands of pounds.

FIG. 2.

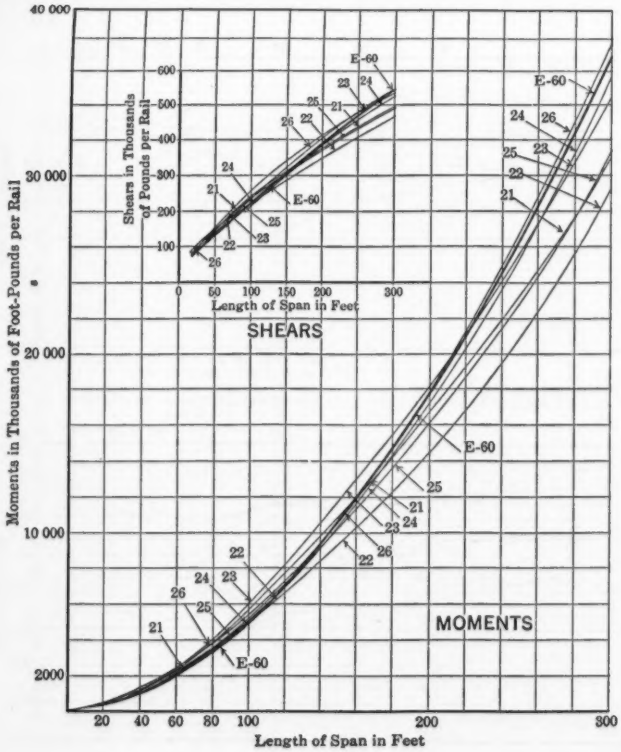

Moments in Thousands of Foot-Pounds per Rail
40 000
30 000
20 000
10 000
2000
20 40 60 80 100 200 300
Length of Span in Feet
MOMENTS
E-60
Shears in Thousands of Pounds per Rail
600 500 400 300 200 100
0 50 100 150 200 300
Length of Span in Feet
SHEARS
E-60

FIG. 3.

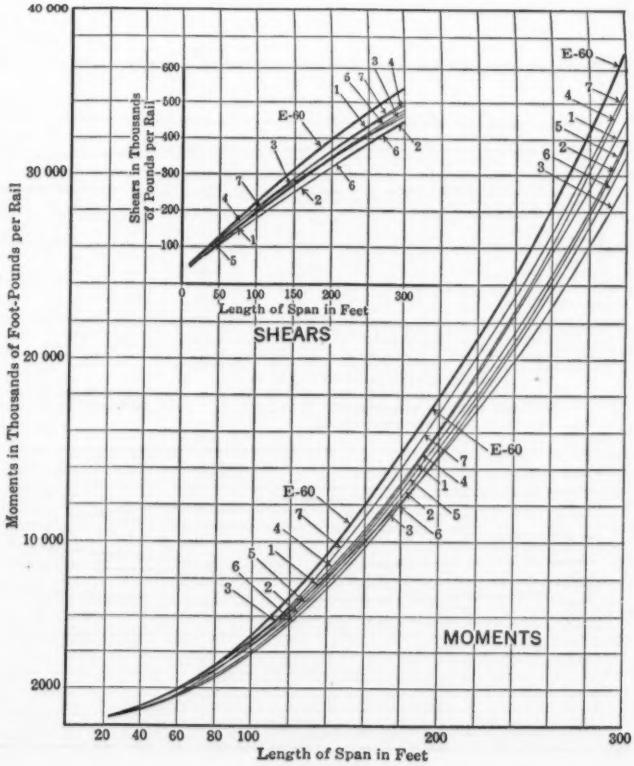
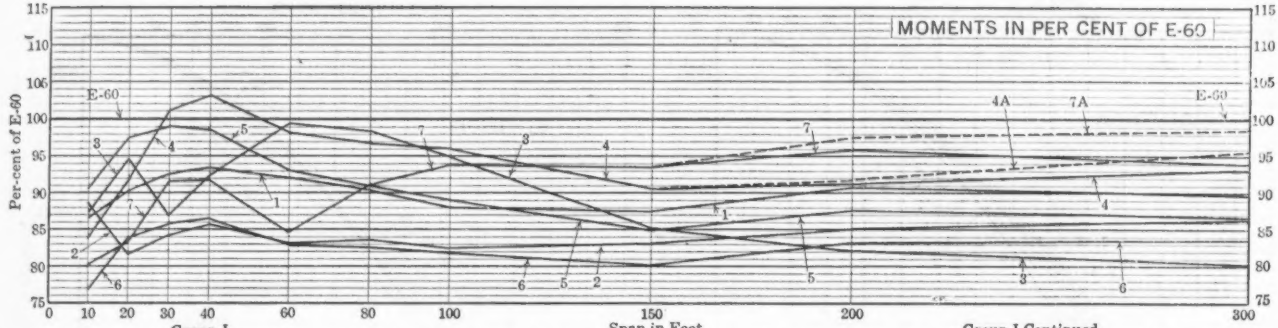

Moments in Thousands of Foot-Pounds per Rail
40 000
30 000
20 000
10 000
2000
20 40 60 80 100 200 300
Length of Span in Feet
MOMENTS
E-60
Shears in Thousands of Pounds per Rail
600 500 400 300 200 100
0 50 100 150 200 300
Length of Span in Feet
SHEARS
E-60

FIG. 4.



Group I

1 U. S. Standard 4-8-2-B & 210 000 lb. cars.
2 U. S. Standard 4-6-2-B & 210 000 lb. cars.
3 U. S. Standard 2-6-6-2-B & 210 000 lb. cars.
4 U. S. Standard 2-10-2-B & 210 000 lb. cars.

4A U. S. Standard 2-10-2-B & 6000 lb. per lin. ft. Uniform Load
7A Santa Fe Type 2-10-2 & 6000 lb. per lin. ft. Uniform Load

Group I Continued

5 N. Y. C. R. R.-Mikado-H7A 2-8-2 & 210 000 lb. cars.
6 U. S. Standard 2-8-2 & 210 000 lb. cars.
7 Santa Fe Type 2-10-2 & 210 000 lb. cars.

Engines 1,2,4,4A,5,6,7, & 7A double-headed; Engine 3 single-headed

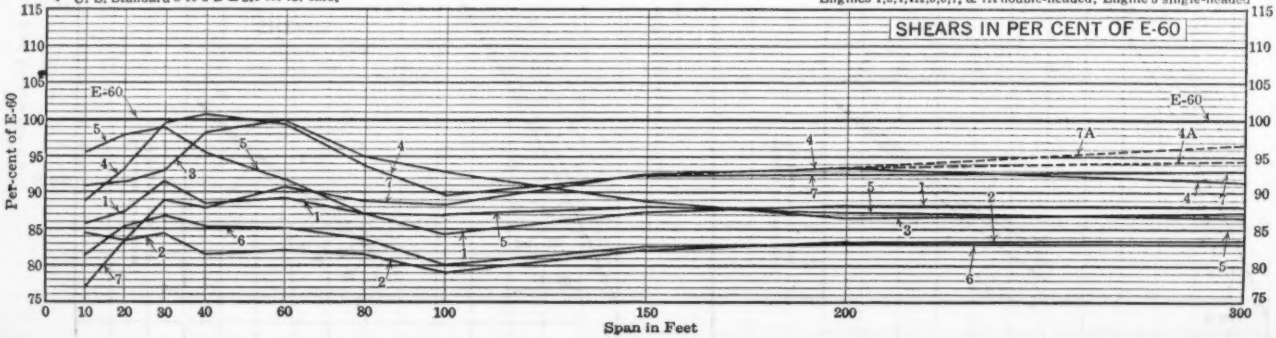
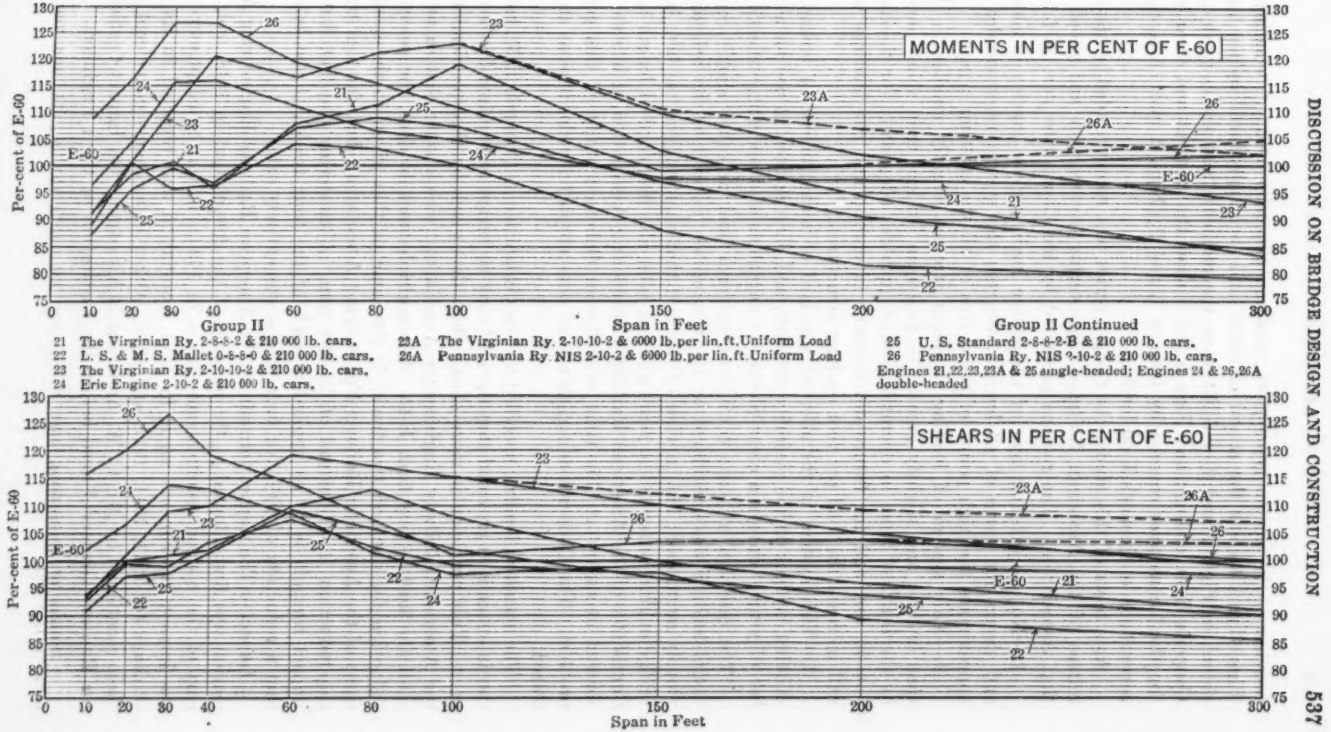


FIG. 5.



Group II

21 The Virginian Ry. 2-8-8-2 & 210 000 lb. cars.
22 L. S. & M. S. Mallet 0-8-8-0 & 210 000 lb. cars.
23 The Virginian Ry. 2-10-10-2 & 210 000 lb. cars.
24 Erie Engine 2-10-2 & 210 000 lb. cars.

23A The Virginian Ry. 2-10-10-2 & 6000 lb. per lin. ft. Uniform Load
26A Pennsylvania Ry. NIS 2-10-2 & 6000 lb. per lin. ft. Uniform Load

Group II Continued

25 U. S. Standard 2-8-8-2-B & 210 000 lb. cars.
26 Pennsylvania Ry. NIS 2-10-2 & 210 000 lb. cars.

Engines 21, 22, 23, 23A & 25 single-headed; Engines 24 & 26, 26A double-headed

FIG. 6.

heaviest of the various types of engines now generally used in special service; Figs. 3 and 4 show the actual moments and shears produced by the loading shown in Figs. 1 and 2, and, likewise, the moments and shears produced by Cooper's E-60 loading; and Figs. 5 and 6 show the moment and shear curves for the locomotives covered by Groups 1 and 2 (Figs. 1 and 2), plotted as percentages of Cooper's E-60 loading.

As a basis of this study, the Cooper series was recommended by the Committee of the American Railway Engineering Association and adopted by that Association. The reasons presented by the Committee were as follows:

1.—No one existing type of locomotive loading gives maximum moments and shears for spans of all lengths, whereas the system recommended does approximate the high points of the curves of all the existing types.

2.—This system of loading produces stresses slightly smaller in short-span bridges and slightly greater in long-span bridges than those produced by the heaviest types in operation, which provides adequately for the engines now in use, and for future development in engine and car loadings which will increase the load per foot on long structures.

3.—This system is the adopted standard of measurement of strength of existing bridges on the majority of roads, and, having been in use for a number of years, conveys a clear picture to engineers and operating officials.

The question of some equivalent uniform load or other simple conventional loading was considered by the Committee, as well as the reasons previously mentioned; but in view of the fact that this problem has been before bridge engineers for many years and has been also discussed at length by the Committee of the American Railway Engineering Association, it was not considered necessary at this time to make an elaborate study of this general problem. The Cooper loading, therefore, was adopted for the reasons already stated.

The standard loading specified by the Committee is Cooper's E-60, but for special cases it is provided that the load may be varied as required by the engineer. This provision is more elastic than that given by the specifications of the American Railway Engineering Association, which state that in no case shall the load be less than E-45. For multiple-track bridges, the percentage reduction for one and two tracks is the same as that given in the specifications of the American Railway Engineering Association; but, for four tracks, the reduction is 30 instead of 25 per cent. Considering the fact that the impact formula is to be applied to each of the four tracks, it was believed that a reduction of 30% for four tracks was entirely safe.

Impact.—The impact formula of the American Railway Engineering Association was first adopted at the meeting of the Association in 1918, and, as a part of the revised specifications, again in 1920. It was based primarily on the impact experiments made by that Association and reported in its *Proceedings* for 1911, 1914, 1917, and 1918. Other experiments considered were those mentioned by Mr. C. W. Anderson in his paper, "On Impact Coefficients for Railway Girders",* read before the Institution of Civil Engineers.

* *Minutes of Proceedings*, Inst. C. E., Vol. CC (1914–15), p. 178, and Vol. CCI (1915–16), p. 301.

In addition to the work of the American Railway Engineering Association, the Committee has also considered the study of the Indian Railway Bridge Committee, 1919-20, and certain English experiments.* The Indian Railway Bridge Committee has been engaged on a series of tests and theoretical studies, but only partial results have been published; thus far, however, they do not indicate any maximum results higher than those of the Committee of the American Railway Engineering Association. The English tests referred to are particularly interesting and valuable because they were made by a recently developed extensometer in which the deformations are magnified by the movement of a mirror and the record is made on a photographic film. This arrangement reduces the inertia of moving parts to a minimum, and should produce a more reliable apparatus, especially for tests on short spans. These tests are also interesting, because many of them were made on very short spans which were not well covered by the tests of the American Railway Engineering Association. The value of some of the published results is considerably affected by the shifting of the zero of the diagram during the test, and because of the great differences between the calculated static stresses and the observed slow-speed values. Comparing these experiments with those of the American Railway Engineering Association, it appears that results for spans of more than 40 ft. are fairly well confirmed by the English tests. On the whole, the English tests show smaller results for these spans than those of the American Railway Engineering Association. For spans less than 30 or 40 ft., the results obtained by the Committee of the American Railway Engineering Association were not generally satisfactory, on account of instrumental vibrations, and in the report of that Committee not much weight was given to results on such short spans. In selecting a formula, the Committee of the American Railway Engineering Association was guided primarily by the results of tests of spans of 30 ft. and more, and for shorter spans by past practice, in which the impact has been taken at about 100 per cent. These considerations resulted in the formula adopted, which gives a reversed curve, starting at 100% for zero span and dropping to 75% for 100-ft. spans. These values are the same as those given by the old formula of the American Railway Engineering Association, and between these span lengths the new formula gives somewhat higher values. For spans exceeding 100 ft., the new formula gives values less than the old one, the difference gradually increasing with the span length.

In discussing the type of formula to be preferred, two main points were considered: First, the proper impact values for short spans, say, up to 25 or 30 ft.; and, second, the proper values for spans longer than about 150 ft. For intermediate lengths, there is little difference between the old and new formulas, and no material change was suggested by any member of the Committee.

For very short spans, the principal results are from the English tests mentioned, which include spans as short as 7½ ft. Two formulas have been

* *Engineering News-Record*, October 20, 1921, p. 642.

proposed by those in charge of these tests, the one preferred being:

$$I = \frac{120}{90 + L}$$

This formula gives 133% impact at zero span and somewhat lower values than the American Railway Engineering Association's curve between spans of 35 and 200 ft., and higher values for longer spans.

Another impact curve which gives a value exceeding 100% for zero span, and which conforms well to the test of the American Railway Engineering Association, is that of Henry B. Seaman, M. Am. Soc. C. E.* This formula is an elliptical curve, tangent to the Y-axis at zero span and to the X-axis at 1 000 ft. span.

Another formula of the same type as the old formula of the American Railway Engineering Association was suggested by a member of the Committee, namely,

$$I = \frac{175}{140 + L}$$

This formula gave an impact of 125% for zero span.

For spans of 200 ft. and more, all experimental results indicate that the old curve of the American Railway Engineering Association is much too high, but naturally there is considerable difference of opinion as to the proper values to adopt.

The desirable type of formula appeared to be dependent on whether or not the impact percentage for zero span was to be made more than 100 per cent. If increased to 125 or 130%, the old type of formula could be made to fit the tests satisfactorily; but if 100% was taken as the value for zero span, then such a curve, if kept high enough for spans of less than 150 ft., would be too high for long spans.

Burton R. Leffler,† M. Am. Soc. C. E. (by letter). For columns, the straight-line formula and the parabolic formula give practically the same average unit stresses. The Rankine formula gives different average stresses. The writer prefers the parabolic formula, if it is necessary to consider a column from a purely empirical standpoint. The parabolic curve fits the tests through the whole range to the point of tangency with Euler's curve. Neither of the other two formulas has this quality; it is necessary to state one or more limits. The writer believes that it is a mistake to consider column tests from a purely empirical standpoint.

It has become the practice to draw a line through plotted results and assume that the equation of the line represents the whole phenomena.

Euler's formula fits the tests for long columns, say, for $\frac{l}{r}$ more than 150, when due consideration is given to the degree of fixation at the ends. A long thin column, such as a sword blade or a steel straight-edge, may fail by pure buckling, with unit stresses below the elastic limit. Euler's formula expresses the law of failure for a straight spring.

* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 318.

† Engr. of Bridges, N. Y. C. R. R., Lines West of Buffalo, Cleveland, Ohio.

A short column fails because the unit stress at some section exceeds the strength, or the elastic limit, of the material. It does not fail because the average unit stress is high. For very short columns the average unit stress is equal to the highest unit stress. At any section of a column there is an axial force and a bending moment. For short columns the bending moment is zero, or nearly zero, depending on how closely the column is centrally loaded.

The problem of the column is one of bending moment. The very fact that columns fail with an average unit stress below the strength of the material is evidence that it is impossible to centralize the load at every section, regardless of how central the load is at the ends of the columns. This is due to the non-homogeneous character of the material, caused by imperfection of manufacture and fabrication. For like reasons a correct theory of the failure of a tension member includes the consideration of a bending moment. Every structural member in a bridge has a bending moment; in tension members its effect is negligible, whereas, in columns, it is important.

At any section of a column the bending moment may be assumed as caused by the load being eccentrically applied. There may be an additional eccentricity due to the load being visibly non-central at the ends of the column.

As soon as an eccentricity is assumed, a correct column formula can be developed. The formula is:

$$p = \frac{f}{1 + \frac{e\,c}{r^2} \sec \frac{l}{2\,r} \sqrt{\frac{P}{E}}}$$

which is known as the secant formula, and is somewhat cumbersome to apply. p is the average unit stress, e the eccentricity, r the radius of gyration, l the length of the column, and c the distance from the gravity axis to the extreme fiber. The other terms are well known. The formula is for columns having knife-edges (not pins) at the ends. For other end conditions, suitable values of l must be used. The reader is referred to the different treatises on mechanics of materials.

All the terms are determinate for the ideal column; but, for the practical column, visibly centrally loaded, e is unknown; it must be determined by experiments. The results of the experiments must be interpreted, keeping in mind the relations of the other terms of the secant formula.

The second term in the denominator of the formula determines that part of f caused by the bending moment.

A column formula which represents the bending fiber stress without the term, c, must be wrong. The distance to the extreme fiber and the radius of gyration are fundamental terms in any formula which purports to give a bending fiber stress.

The secant formula compels the designer to consider the section of a column in a thorough manner. It is interesting to note that Tredgold and Gordon considered the c term to be important in a column section. Deter-

mining the area of a column section should be something more than a blind use of $\frac{l}{r}$ in a formula.

The writer regards the tests of columns as useless until engineers rationalize the results. The form of the section and the difference between a purely rolled column and a built-up column should be given more study.

Where a problem admits of a partial rational treatment, it seems wrong to solve it in a purely empirical way. Engineers should couple rationalizing and experimenting.

Let us consider further the formulas: Take $\frac{l}{r} = 40$. The fiber stress due to bending is 2 000 lb. for the straight-line formula, 400 lb. for the parabolic formula, and 1 700 lb. for the Rankine formula. As the straight-line and parabola almost coincide, why the difference? The writer will leave this to the reader to think about. If the shear for column lacing is calculated, marked differences will also be found.

The secant formula includes Euler's formula. Arthur Morley deduces the formula by a substitution of terms in the chain of reasoning leading to Euler's formula.*

The secant formula fits the results of tests closer than any other.†

The advocates of the Rankine formula claim that it is rational, but how can it be with the c term omitted? The late J. B. Johnson, M. Am. Soc. C. E., deduced the formula‡:

$$p = \frac{f}{1 + \frac{e\,c}{r^2} + \frac{f - p}{10\,E}\left(\frac{l}{r}\right)^2}$$

The formula is based on the bending diagram as being a parabola; for the secant formula, the diagram is a sinusoid. Rankine's formula is derived from Johnson's formula by omitting the second term of the denominator and substituting an empirical constant for the coefficient of the last term. Such transformation is not valid; it shows Rankine's formula to be strictly empirical. On account of this transformation, the formula does not fit test results as well as the parabolic formula.

A riveted column should be designed for a section less than the gross. Riveted tension members are designed for net section. Probably a somewhat less allowance would do for columns. Riveting should be reduced to a minimum, and should be symmetrical at any section of the column. The ordinary column composed of two channels connected by single lacing is unsymmetrically riveted.

Symmetry should be a cardinal principle in column design, it reduces the unknown eccentricity; minimum fabrication is also conducive to this end.

Francis P. Witmer,§ M. Am. Soc. C. E. (by letter).—In Fig. 7, curves are shown for the following column formulas:

* Morley's "Strength of Materials", Article 104.
† "Modern Framed Structures", Pt. 3, Article 43.
‡ *Loc. cit.*, Article 134 of Fourth Edition.
§ Cons. Engr., New York City.

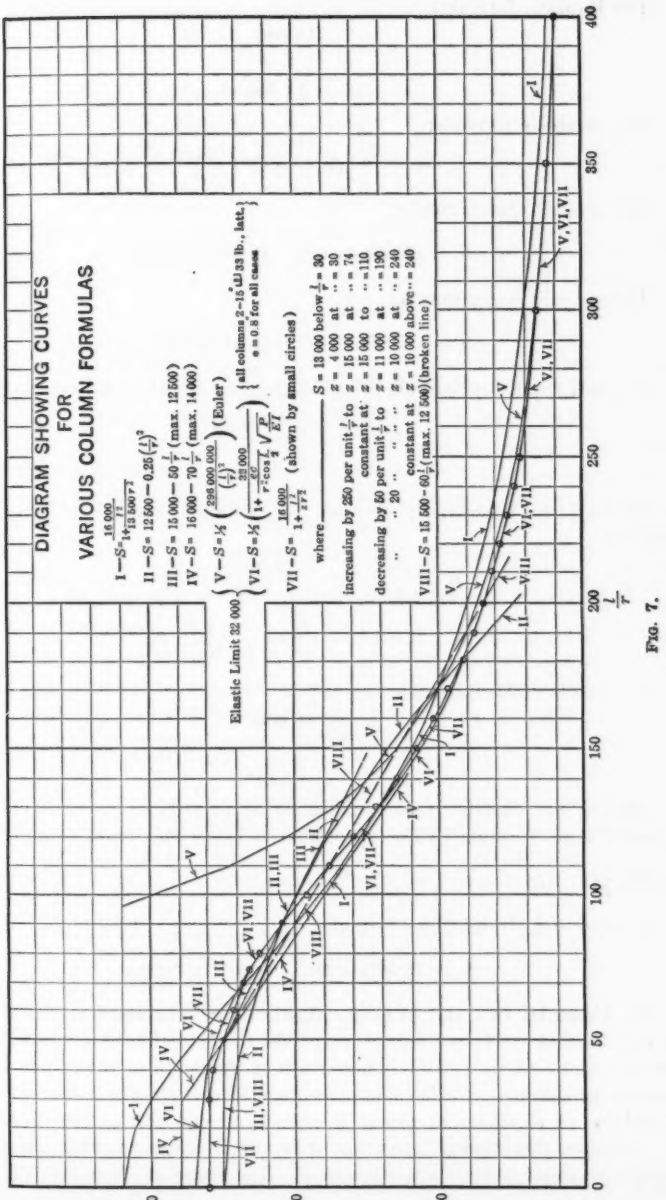

DIAGRAM SHOWING CURVES
FOR
VARIOUS COLUMN FORMULAS
I — $S = \frac{16\,000}{1 + \frac{l}{12\,500\,r^2}}$
II — $S = 12\,600 - 0.25\left(\frac{l}{r}\right)^2$
III — $S = 15\,000 - 50\frac{l}{r}$ (max. 12 600)
IV — $S = 16\,000 - 70\frac{l}{r}$ (max. 14 000)
Elastic Limit 32 000
V — $S = \frac{1}{2}S\left(\frac{295\,000\,000}{\left(\frac{l}{r}\right)^2}\right)$ (Euler)
VI — $S = \frac{1}{2}S\left(1 + \frac{295\,000}{r^2 \cos\frac{e}{2}\sqrt{\frac{P}{EI}}}\right)$ {all columns 2–15 lbs. 33 lb., latt.; e = 0.5 for all cases
VII — $S = \frac{16\,000}{1 + \frac{l^2}{r^2}}$ (shown by small circles)
where S = 13 000 below $\frac{l}{r}$ = 30
increasing by 250 per unit $\frac{l}{r}$ to $\frac{l}{r}$ = 74 at S = 4 000
constant at S = 15 000 to $\frac{l}{r}$ = 110
decreasing by 50 per unit $\frac{l}{r}$ to $\frac{l}{r}$ = 190 at S = 11 000
" 20 " " " = 240 at S = 10 000
constant at S = 10 000 above
VIII — S = 15 500 − 60 $\frac{l}{r}$ (max. 12 500) (broken line)
Allowable Unit Stress "S" in Pounds per Square Inch
15 000
10 000
5 000
0
50
100
150
200
250
300
350
400
$\frac{l}{r}$
I
II
III
IV
V
VI
VII
VIII
V, VI, VII

FIG. 7.

The Rankine formula:

$$S = \frac{16\,000}{1 + \frac{l^2}{13\,500\,r^2}} \quad \text{.....(I)}$$

The parabolic formula:

$$S = 12\,500 - 0.25\left(\frac{l}{r}\right)^2 \quad \text{.....(II)}$$

The straight-line formula:

$$S = 15\,000 - 50\,\frac{l}{r} \text{ (max. 12 500)} \quad \text{.....(III)}$$

The straight-line formula:

$$S = 16\,000 - 70\,\frac{l}{r} \text{ (max. 14 000)} \quad \text{.....(IV)}$$

One-half the Euler formula for ultimate strength of round-ended columns:

$$S = \frac{1}{2}\left(\frac{296\,000\,000}{\left(\frac{l}{r}\right)^2}\right) \quad \text{.....(V)}$$

One-half the eccentricity formula for ultimate strength of round-ended columns:

$$S = \frac{1}{2}\left(\frac{32\,000}{1 + \frac{e\,c}{r^2 \cos\frac{l}{2}\sqrt{\frac{P}{E\,I}}}}\right) \quad \text{.....(VI)*}$$

A formula of Rankine type:

$$S = \frac{16\,000}{1 + \frac{l^2}{z\,r^2}} \quad \text{.....(VII)}$$

in which z has variable values, taken so as to produce a curve practically identical with Formula (VI), except that S has a maximum value of 13 000 (below $\frac{l}{r} = 30$).

A suggested straight-line formula:

$$S = 15\,500 - 60\,\frac{l}{r} \text{ (max. 12 500)} \quad \text{.....(VIII)}$$

For Formulas (V) and (VI) the elastic limit is assumed at 32 000 lb. per sq. in. Formula (VI) not only corresponds, theoretically, to a constant initial eccentricity of 0.8 in. (which is about 5% of the depth of the column), but it can also be shown that this same percentage of eccentricity is consistent with a modulus of elasticity of nearly $E + 10\%$, on one side of the column, and $E - 10\%$, on the other side, so that it is also in harmony with a not unreasonable assumption as to variation in E. The curve of Formula (VI) agrees

* In formula (VI), all columns are assumed to consist of two 15-in. channels, 33 lb., latticed, and the eccentricity, e, is assumed at 0.8 in. for all cases.

as well with test results as Formulas (I), (II), (III), or (IV), and has the additional merit of being continuous for all values of $\frac{l}{r}$, blending into Euler's curve for the higher values. The only valid objection to its adoption is the inconvenience of applying it. Formula (VII), therefore, has been devised to give practically the same curve as Formula (VI), but to have the advantage of the Rankine formula as to ease of application, except that the coefficient, z, of r^2, instead of being constant, varies according to a simple law.

If objection is raised on theoretical grounds to a variable coefficient of r^2 in a formula of the Rankine type, it may be met by the argument that the Euler formula, itself, may readily be transformed into a Rankine formula, in which z varies from 13 500 at about $\frac{l}{r} = 171$ to 9 250 at $\frac{l}{r} =$ infinity. Why, then, should not a variable value be considered reasonable for lower length ratios, where Euler's formula is not applicable?

A little consideration will show that the column shear due to assuming the bending to be caused by an initial eccentricity, is less than that due to assuming it to be caused by an initial deflection, and, therefore, to be on the safe side, the latter assumption should probably be made in calculating shearing stresses, although at some sacrifice of consistency. With a column formula of the Rankine type, this assumption will lead to a shear having the value, $V = \frac{\pi P L}{z c}$ (if the curve is assumed to be a sinusoid), P being the total load, L the length, c the distance from the neutral axis to the extreme fiber, and z the coefficient of r^2 in the Rankine formula. Hence, Formula (VII) may be consistently used for calculating column shears.

It cannot be denied that some formula would be desirable, that had a rational form and that would be applicable to higher values of $\frac{l}{r}$ than the limiting ones prescribed in the specifications. Although adherence to these values admittedly would control ordinary bridge design along safe and proper lines, yet there is no doubt that any formula recommended by the Society will be applied on occasion to higher values than those prescribed, as, for example, in investigating the strength of existing structures, or in the design of special structures, such as transmission towers; and the formula recommended should not lead to results either dangerous or unduly on the safe side. Formula (II) would produce the latter result, and Formula (III), the former, if pushed to extremes. Formula (I) would be fairly conservative, but its curve lies considerably above that of Euler for higher length ratios, and this makes its use in such cases questionable. Undoubtedly, Formula (VI), or its simpler equivalent, Formula (VII), would be preferable, if it were not considered too difficult of application.

If a straight-line formula was adopted finally, it would seem that it should lie somewhere between Formulas (III) and (IV), as, for example, a line passing through $S = 12\,500$ at $\frac{l}{r} = 50$, and drawn tangent to Euler's curve, V.

Formula (VIII) is suggested as approximating these conditions. Its results would not differ materially from those of Formulas (I), (II), or (III), for ordinary length ratios, and it could safely be used up to $\frac{l}{r} = 200$, as it does not depart materially from Euler up to that point.

In conclusion, the writer wishes to emphasize once more the suggestion that the formula adopted by the Society should be one which can be properly used by engineers, with a minimum of reservations as to its applicability, because it may be safely predicted that it will be extensively used on its own merits, entirely apart from the general body of the specifications, in view of its authoritative source, and, therefore, it should not be in any degree misleading.

JAMES E. HOWARD,* ESQ. (by letter).—Referring to the subject of column formulas, it appears to the writer fundamentally desirable to restrict the length of columns when feasible to do so. The failure of a column, if it occurs, is likely to be the result of an emergency, such as being called on to act as a collision strut.

The manner of failure in the last stages of long and short columns differs. A period is reached in the failure of a long column when sudden lateral deflection, or springing, occurs, attended with great loss in resistance. If, perhaps, a long column was forced out of line a certain distance, collapse of the structure would certainly follow. A sudden drop in sustaining power is not characteristic of a short column.

Concerning column formulas, the writer has been forced to take a position at variance with prevailing views, raising the question whether, in practice, they are what they purport to be.

An early series of tests on Phœnix columns was made by the writer, the results of which have been reported to the Society by Messrs. Clarke, Reeves, and Company.† Certain queries arose during the making of these tests, which led to making another series soon thereafter on plain square bars. The latter measured 3 in. on a side, were tested chiefly with pin-ends, and were of different ratios of slenderness.

At the time of these tests, few modified column formulas had been presented, those of Euler, Rankine, or Gordon being in use. A feature was brought out in the tests of the square bars, not peculiar to these small members, but common to structural shapes, which seemed to militate against any formula except a purely empirical one, namely, the presence of a state of internal strain which arose from two causes, the cooling strains of fabrication, and those set up by cold straightening. Although the investigation of these internal strains was not undertaken until some years later, when numerical values could be assigned them, nevertheless the importance they played was then quite evident.

It appeared subsequently that internal strains from these two causes differed somewhat. Strains from cold straightening were more likely to influence

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† "Experiments upon Phoenix Columns", *Transactions*, Am. Soc. C. E., Vol. XI (1882), p. 1.

the results of compression tests than cooling strains, considering each separately, and as applied to long columns. The reason for this was that internal cooling strains might and probably would leave a substantial margin between the value of the elastic limit of the metal and their own magnitude. This margin would represent the external stress that the member could endure without taking a permanent set, that a state of residual perfect elasticity of some degree, would remain in the member.

Cold straightening is an overstraining of the metal, in which there is impairment of the elastic limits of tension and compression, in addition to which internal strains are set up by the resilient action of the metal when the straightening force is removed. These internal strains are left in a state of equilibrium, but delicately poised, in which their relations to each other are easily disturbed.

Having these features in mind, it appeared that the elastic limit of a column would vary in different parts of its cross-section, and, as early permanent sets would develop locally, that the strength of a long column in particular would be affected by the state of internal strain of its component parts. Here was a factor, it seemed, that might have an influence on the strength of the column of such degree that the application and use of a formula would be nullified.

Internal strains have no influence on the modulus of elasticity of the metal, except that overstraining is known to lower its value temporarily, but from which there is early recovery. The presence of internal strains leads to the display of early sets, perhaps premature local buckling, and local yielding has its effect on the ultimate strength of compression members. Failure in this manner is not taken into account in a column formula.

Internal strains equivalent to working stresses, according to current practice, can hardly be regarded with indifference in their relation to a column formula, and, not being taken into account, may assist in explaining the recurrence of the discussion of the subject.

CROSBY MILLER,* ASSOC. M. AM. SOC. C. E. (by letter).—The Society should consider the development of a specification using a unit tensile figure of about 23 000 lb. per sq. in., and a live load that would represent the greatest which, in the opinion of the designer, would ever pass over the structure, this live load to be expressed in the Cooper loading. A specification of this type will result in a saving in material, due to the use of a higher unit stress for dead load, and, where no future load greater than E-60 is anticipated, would allow the use of a lighter structure.

C. D. PURDON,† M. AM. SOC. C. E. (by letter).—The following suggestions in regard to specifications for steel railway bridges are offered. The width, center to center, of trusses, should not be less than one-twentieth of the span.

Where there is much snow, the guard-rail bolt should be countersunk flush with the top of the guard-rail; a snow plow may strike it.

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† Cons. Engr., St. Louis Southwestern Ry., St. Louis, Mo.

Why not take the actual load per foot for ballasted floors? The alternate load of 75 000 lb. on each of the axles on a 10-ft. span equals 15 000 lb. per ft. as a maximum.

The writer thinks it would be better to specify E-10 and to make the lower limit E-45. For many branch lines, engines of about E-25 loading are ample for the loads they have to haul, and E-60 means hauling so much more dead load. The limit of E-45 would take care of such cars as the 50-ft. gondolas of 70 tons capacity, of the Pennsylvania Company.

As the coefficient of expansion for steel is 0.0000065 per degree Fahrenheit, it would seem that the coefficient of the American Railway Engineering Association of 1 in. per 100 ft. is ample.

In connection with impact, it has been stated that some English experiments gave 125% on a length of 7 ft. 6 in. The alternate loading of 75 000 lb. on each of two axles, 7 ft. 6 in. apart, gives more than the regular loading, 25% on a 5-ft. length, reducing to 11% on a 10-ft. length; this, with the impact formula of the American Railway Engineering Association, would give an impact of 125% on 5 ft.

This alternate loading does not affect a length of more than 11 ft. The fact that the American Railway Engineering Association specification is limited to spans of less than 300 ft. seems to have been overlooked.

G. B. Woodruff,* Assoc. M. Am. Soc. C. E. (by letter).—The following suggestions as to detailed specifications for steel railway bridges, are offered:

Rolled beams can be used to advantage up to spans of 35 ft.

In order that the main-track alignment may be kept straight, it is often desirable to build four-track bridges with four trusses.

With heavy rails (135-lb.) and guard-rails, rails and fastenings will weigh more nearly 200 lb. per lin. ft. of track.

The writer believes that economy of design requires the use of a heavy live load (say, Cooper's E-90) and of increased unit stresses. A 50% increase in live load would increase the unit stresses in stringers, hangers, and some web truss members about 50% above the specified values, while the chord members of heavy bridges would be overstressed, say, only 25 per cent.

The lateral force should be 700 lb. per lin. ft., as it is simple to calculate, and will give satisfactory results.

The writer believes that provision for an expansion of 1 in. for every 100 ft. in length is sufficient. He is familiar with a number of bridges where the expansion bearings have been marked to determine the expansion, and believes the figure given is more than sufficient.

It may be called "practicable" to use 8 by 8 by 1-in. angles, with no covers, for a girder flange. The specification that "preferably the angles compose 50% of the flange section and in no case shall the flange cover-plates compose more than 60%", is more definite.

Flange splices are necessary in very long girders, and the writer believes that this specification should be extended as in those of the American Railway Engineering Association.

* Asst. to Bridge Engr., Lehigh Valley R. R., Bethlehem, Pa.

Rules for the rating of old structures have been published recently, which allow a unit stress of 26 000 lb., with secondary stresses not considered. It would seem that the margin between this and a 30 000-lb. yield point is very small. Some railroads are now specifying a 33 000-lb. yield point and practically the same elongation and bending tests.

The writer does not believe that full punched work, for main sections, has any place in a present-day specification for railroad bridges, especially if that metal is to be stressed to 26 000 lb. per sq. in., and considering that, in the bridge shop, workmanship is subordinated to tonnage.

C. E. Conover,* M. Am. Soc. C. E. (by letter).—Relative to bridge design for electric traction: From all information that has come to the writer's attention, cars equipped with direct drive produce but little impact, probably not more than 10% for ordinary speeds. As a matter of fact, probably most of that comes from imperfect track or wheel conditions. It is believed that the impact for such bridges could be reduced so as to be not more than 25 per cent.

In regard to depth and spacing of girders, the writer would suggest for bridges for electric railway operation, with cars operated by direct-drive motors, that the girders might as well be 5 ft., center to center, reducing the strain on the ties. It is his judgment, based on information that has come to his attention, that the impact in the case of such cars is very small. There seems to be little necessity for attempting to eliminate the direct effect of this impact by not having wheels directly over the girders.

Edward Godfrey,† M. Am. Soc. C. E. (by letter).—The impact for bridges to carry electric traction exclusively, is required to be one-third as great as for railroad bridges. In the writer's judgment, an impact allowance of 25% for all members of a traction bridge is sufficient. Impact is due almost, if not, entirely to roughness of the track or unbalanced wheels. The effect of impact on any member has little or no relation to the length of span or loaded portion; it depends on the imminence of the member to the track. In very short spans, the member or girder is naturally closer to the track, and tests show higher impact. There is nothing in test results of impact observations to justify a formula that varies the impact with the span length. The only rational method of taking care of the dynamic effect of rolling loads is one on the order of Cooper's specifications, in which the effect of live load on a member is varied in accordance with the directness of its action on that member. In the matter of traction bridges, it seems that the standard practice of using an impact of 25% of the live load on all members should be used.

If one sits on the bottom chord of a bridge while a 5-ton automobile passes at a great speed over the bridge, and from the same location compares the results with what happens when a horse, pulling a light wagon, trots across, one will be convinced that the suddenness of application of the load has little

* Designing Engr., Transit Comm., New York City.
† Structural Engr., Pittsburgh, Pa.

to do with the dynamic effect, whereas vertical vibration, due to the roughness of the floor or the track, or any other cause, has a large influence.

No column formula can be even approximately correct that fails to take into account imperfections in columns, this is particularly true for structurally permissible columns. Years ago, engineers observed that, in the zone of structurally permissible columns, test results were grouped about a practically straight line and then curved up in the zone of slender columns. On the strength of this, the straight-line formula for columns was urged and, later, generally adopted.

This straight-line formula is suitable because it not only is simple of application, but it also virtually recognizes imperfection in columns, as it comes nearer to an agreement with the strength of commercial compression members than any other formula in use.

Among engineers who favor theory, there seems to be a movement to return to the curved-line formulas. The chief argument for the curved-line formula is the inapplicability of the straight-line formula to slender columns—columns outside the range of structural specifications. If the straight-line formula was shown to give false results within the limits where it is used, this would be an argument against it; but because it fails to give proper results in a field where the use of compression members is forbidden, is no reason for its abandonment.

A formula that would give a "whip-shaped" curve of column unit stresses, that part in the region of permissible structural columns being straight and that in the "slender" zone curving up, would agree best with test results. A curve of this shape would naturally have a complex equation, and its use would impose an unnecessary burden on the computer.

That test results do group around a "whip-shaped" curve has been shown in results published in an article* which is a plea for the revival of the Gordon-Rankine formula.

There is no reason why the straight-line formula should not be used for columns up to the maximum ratio of slenderness permitted in structures, slender compression members, such as posts for transmission towers and struts for aeroplanes, being designed by a formula appropriate to that used in the design of long struts. The Euler formula is the proper formula for this purpose, but it is useless for designing structurally permissible columns.

Tests demonstrate that steel is not as good in compression as in tension. It is not clear why the user of the Gordon-Rankine formula should be allowed a maximum unit stress of 14 300 lb. per sq. in., whereas the user of the straight-line formula is allowed a maximum of 12 500 lb. per sq. in. There is no connection between the type of formula used and the maximum unit stress allowed. It seems well to adopt an upper limit of 12 500 or 13 000 lb. per sq. in.

The unit pressure allowed on concrete masonry is 600 lb. per sq. in., which gives an undue advantage to concrete over limestone masonry which is allowed 400 lb. per sq. in. Bridge seats are seldom grouted or embedded in a manner

* *Engineering News-Record*, March 10th, 1921, p. 431, Figs. 1, 2, and 5.

that would give a true bearing, and for this reason unit pressure should be kept low.

Secondary stresses and bending moments due to deflection of floor-beams may be safely ignored in ordinary structures; in fact, it is doubtful whether they need ever be considered.

Specifications should contain a definite pronouncement on the subject of tension on rivet heads. Inhibition of tension on rivet heads defeats its own purpose, whether that purpose is to secure good design or to prevent such tension. Tension on rivet heads gives rigidity and stability to all riveted structures. Without it, structures would rattle and fall apart; it is this tension that supplies the friction that plays a major part in holding riveted pieces together.

Hardly a design can be made that does not involve some direct tensile stress on rivet heads. Rivets are capable of taking a large amount of tension on the heads and, with proper design, of doing so with safety.

On this subject the following is recommended:

"Tension on rivet heads will be allowed only when the connection is symmetrical and contains at least four rivets. The bending at the heel of connection angles must be considered, as well as the bending on the part to which the detail is attached. When $\frac{7}{8}$-in. rivets are used, the connection angles should be $\frac{5}{8}$-in. metal and the gauge of rivets 2 in. With $\frac{3}{4}$-in. rivets use $\frac{1}{2}$-in. angles and $1\frac{3}{4}$-in. gauge. Additional rivets must be provided to take any shear coming on the same connection."

The writer made a series of tests* which seemed to show clearly that the minimum net area of a bar, measured on a zigzag line, joining rivet holes, is the measure of the value of the bar in tension. Opportunity was afforded to bring evidence to controvert this, but no such evidence was furnished. Tests on riveted seams were cited, but a riveted seam in which the stress originates in the rivets, is quite different from a bar or flat, in which the stress is axial and merely distributes itself around holes cut out of the section.

D. B. Steinman,† M. Am. Soc. C. E. (by letter).—In fixing unit stresses for design, the safe stress for dead load should govern. A basic unit stress of 20 000 lb. per sq. in. would leave ample margin below the elastic limit, and would unquestionably be safe for dead load. Providing for dead load stress at 16 000 lb. per sq. in. is a waste of metal. Years ago, Cooper established 20 000 lb. per sq. in. as the working stress for dead load, and the advance since then in the structural art should justify a reduction and not an increase in the "factor of ignorance."

The difference between the effect of live load and that of dead load should be taken care of by the impact allowance and not by a reduction in working stress. If 20 000 lb. per sq. in. is a safe stress for dead load, it is also a safe stress for live load plus impact.

The provision for future increase in live load should be made by assuming a heavier live load, and not by reducing the working stress. To provide for

* *Engineering News*, May 3d, 1906.

† Cons. Engr., New York City.

probable increase in loading by using a reduced working stress is illogical, and results in an unbalanced design. The live load may increase, but the dead load will not; as a result, the bridge members carrying the largest proportion of dead-load stress will be the most extravagantly designed, and the members carrying the largest proportion of live-load stress will be relatively the most deficient in section. It cannot be disputed that exactly the reverse condition ought to be the objective in proper design. To secure the largest possible margin of strength for increasing loads, with a minimum expenditure of metal, the extra metal should be distributed among the various members in proportion to their live-load stresses. Accordingly, the most economical and best balanced design will be secured by adhering to the principle previously enunciated, namely, the provision for future increase in live load should be made by designing for a heavier live load, and not by reducing the working stresses.

The writer maintains that the working unit stress in tension should be at least 20 000 lb. per sq. in. instead of 16 000 lb., and that the other unit stresses should be increased in the same ratio.

Live Load.—It is disappointing to have the important question of live load overlooked, and to retain the ancient Cooper loading in the proposed specifications.

The Cooper loading does not correspond to modern locomotives, and its retention as a standard for design is unscientific. Its use produces designs that are unbalanced for modern loadings. For instance, the Pennsylvania Railroad N-1-S locomotives are equivalent to Cooper's E-55 for some bridge members and to Cooper's E-76 for others, a range of variation of 38 per cent. The Erie 2-10-2 locomotives are equivalent to Cooper's E-53 for some bridge members and to Cooper's E-70 for others, a range of variation of 32 per cent. Other locomotives, compared by the writer, gave similar ranges of variation. In other words, structures designed for the arbitrary Cooper loading will not be of uniform strength for a given modern locomotive; some of the members in a span will be defective in strength, and some of the spans in a line will be disproportionately weak, in comparison with others, and this difference in strength may be as much as 30 or 40 per cent. A loading that yields such disproportionate results may well be regarded as obsolete. A Cooper engine diagram does not represent modern locomotives, and should not be used for designing modern structures.

An argument advanced for retaining the Cooper loading is that it affords a convenient standard for rating bridges; but the writer maintains that a standard that has become warped to such an extent as to yield results that fail to harmonize by 30 to 40% has outlived its usefulness. A loading should satisfy the following requirements:

1.—It should give results that harmonize closely with the stresses producible by modern heavy locomotives.

2.—It should be heavy enough to provide for the probable future increase in loading during the desired life of structures. Accordingly, it should be

25 to 50% heavier than the average present loading (supposed to be represented by Cooper's E-60).

3.—It should be presented in such form as will yield maximum facility of application and will discourage meaningless refinement of computation.

Impact.—In past American practice, there have been two different types of impact formulas:

1.—Formulas based on the ratio of live-load stress to live plus dead-load stress, $\frac{L}{L+D}$.

2.—Formulas based on the length (l) of load producing the stress.

The writer is convinced that formulas of either class are only partly correct; and that, to be truly scientific and comprehensive, an impact formula should combine both the factors enumerated. In other words, a term of the form, $\frac{L}{L+D}$, should be incorporated in the impact formula.

The inclusion of this factor will make automatic correction for the greater inertia resistance of the heavier sections in a given structure and of the heavier structures in a given line. It will apportion the largest relative additional metal for impact to those members in which the live load bears the largest ratio to the dead load, thus contributing to the longevity of the structure under increasing loads. It will yield results conforming to the principle that impact should be less on bridges with solid ballasted floors than on those with open decks. It will yield a reduced impact for bridges carrying a highway floor in addition to the railway tracks. It will be a partial compensation or incentive for adopting the heavier and more rigid construction in a choice between alternative designs. It will automatically increase the impact tax on very short spans and reduce it on very long spans. Moreover, it will yield a formula applicable without change to all types of structure on a line, instead of requiring different formulas to be used for steel trusses, concrete arches, etc.

The formula recommended in the Tentative Specifications is defective in that it makes no distinction between light and heavy constructions of the same length of span; for instance, it would yield the same impact stresses for bridges with solid decks as for those with open floors. This defect would be remedied, and the other advantages mentioned previously would be secured, if the impact formula was corrected to include the factor, $\frac{L}{L+D}$.

The revised formula, moreover, should conform with observed effects by yielding impacts exceeding 100% for very short spans. Accordingly, the writer would propose the following formula for impact percentages so as to combine the two contributing factors of load length and load ratio:

$$I = \frac{200}{100 + l} \times \frac{L}{L + D}.$$

The values yielded by this formula for open-deck spans of various lengths conform closely to the maximum impact percentages recorded in the American and the English tests. This is shown by Fig. 8, where the graph of impact values calculated by this formula (for open-deck spans), practically coincides with the highest test values recorded from all lengths of spans. For solid-deck spans, the impact values yielded by this formula are appropriately lower. The formula is simple and easy to apply.

Column Formula.—The conditions do not warrant hair-splitting precision in fixing working stresses for columns, and there seems to be little justification for using quadratic formulas when a simple straight-line formula gives essentially the same results. A straight-line formula with a horizontal maximum should suffice.

One important point that has not been provided for in any of the standard specifications, is the difference in permissible stress in columns having different forms of open or closed sections. Closed or box sections are safer against buckling than open sections, and solid diaphragms or cover-plates are more effective than planes of latticing. This difference in value of different forms of section should be represented in the column specification. The column formula should allow a higher working stress for the sections that are known to be safer, so as to offer a premium for the selection of the more substantial designs, instead of encouraging, as the present specifications do, the adoption of open, flaring sections for the sake of economy.

On the basis of a working stress of 20 000 lb. per sq. in. for tension, the writer would recommend that the following working stresses be specified for columns:

For a closed section, or a section with two diaphragms, or a section with one diaphragm and two planes of lattice:

$$22\,000 - 100\,\frac{l}{r} \text{ (maximum, 20 000).}$$

For a half-open section with one cover-plate and one plane of lattice, or a section with one diaphragm without latticing:

$$20\,000 - 100\,\frac{l}{r} \text{ (maximum, 18 000).}$$

For an open section with two or more planes of lattice:

$$19\,000 - 100\,\frac{l}{r} \text{ (maximum, 17 000).}$$

The writer maintains that the adoption of such specifications for column design will be conducive to better and more consistent designing.

If, in addition, it is desired to include in the column formula the extreme fiber distance, c, as urged by Mr. Leffler,* the writer would suggest that the term, $100\,\frac{l}{r}$, in the formulas previously mentioned, be replaced by a term:

$$80\,\frac{l}{r}\,\frac{c}{r}.$$

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* See p. 542.

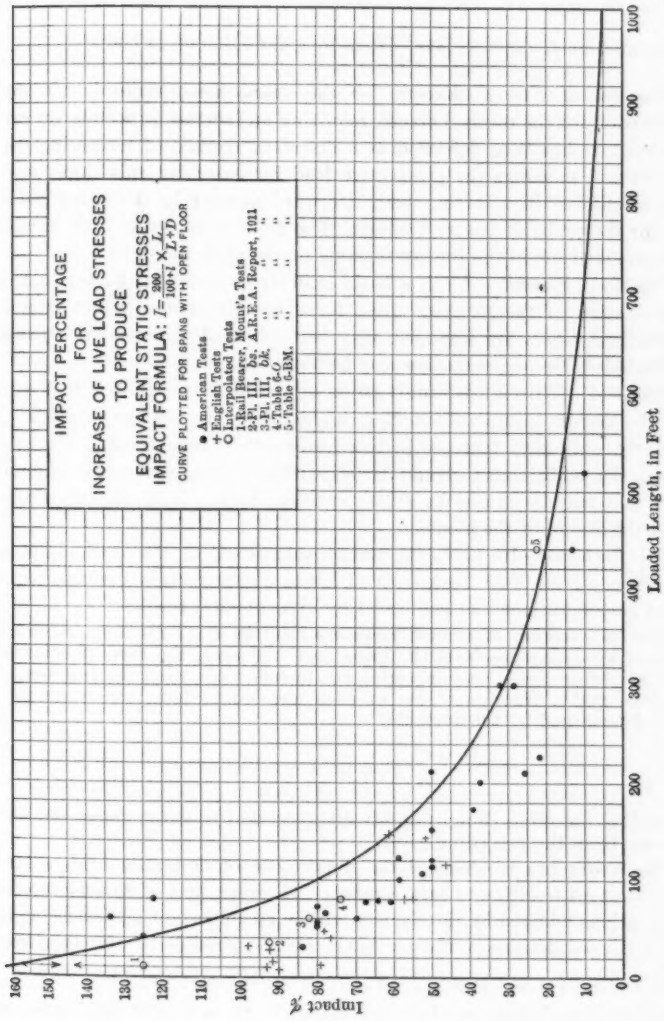

IMPACT PERCENTAGE
FOR
INCREASE OF LIVE LOAD STRESSES
TO PRODUCE
EQUIVALENT STATIC STRESSES
IMPACT FORMULA: I = 200/(100+L) × L/(L+D)
CURVE PLOTTED FOR SPANS WITH OPEN FLOOR
● American Tests
+ English Tests
○ Interpolated Tests
1-Rail Bearer, Mount's Tests
2-Pl. III, 5a, A.R.E.A. Report, 1921
3-Pl. III, 5b, " "
4-Table 6-C, " "
5-Table 6-BM, " "
Impact %
Loaded Length, in Feet

FIG. 8.

Spacing of Trusses.—Past practice has required a width between centers of trusses of about one-sixteenth of the span, and it seems to be a step in the wrong direction to reduce the prescribed width to one-twentieth of the span.

Pin Stresses.—The safe bending stress in extreme fibers of pins is better represented by $40\,000 - 1\,000\,\frac{l}{d}$, than by a flat value of 24 000.*

Reversal of Stress.—Except for the connections, there is no physical justification for the reversal clause, requiring an increase in section for reversal of stress. It has been defended as a disguised method of increasing the sections of web members for prospective load increase; but this object ought to be accomplished in a direct, straightforward manner by designing the structure for the probable future loading. The greater impact on web members is fully provided for in the impact formula proposed by the writer.

Combined Stresses.—With a basic unit stress of 20 000 lb. per sq. in., the augmentation of the working stress for a combination in which lateral and longitudinal forces are included, may be 20%; and if secondary stresses are also included, the augmentation may be 30 per cent.

Overload.—With a basic unit stress of 20 000 lb. per sq. in., it is assumed that the design load is about 25% higher than the present loadings. Consequently, only 25% increase in live load need be considered in the overload clause. In addition, it should be stipulated that any reversal of resultant stress that may be caused by a 25% increase in live load, should be provided for in the design of the member.

Net Sections.—In 1920, the writer submitted, to the Bridge Engineers' Committee of the New York Central Lines, a specification derived from theoretical considerations and confirmed by experimental results:

"The net section of riveted members shall be the least area which can be obtained by deducting from the gross sectional area, the area of holes cut by any straight or zigzag section across the member, counting the full area of the first hole and a fractional part of each succeeding hole, the fractional part being determined by the formula,

$$1.3 - \frac{P}{G} \text{ (but not to exceed unity)},$$

where P is the pitch (longitudinal) distance, and G is the gauge (transverse) distance from the preceding rivet."

The writer has emphasized the importance of designing bridges for ultimate loads with ultimate safe stresses. The retention of this feature in the specifications adopted by the Society will be an important achievement; it will yield balanced designs conducive to the longest possible life for the structure.

PERCIVAL S. BAKER,† M. AM. SOC. C. E. (by letter).—The Pencoyd formula, which has been largely used during recent years, increases the impact only a small amount on spans of less than 100 ft., but on spans of more than 100 ft.

* See the writer's article, "Higher Stresses for Bridge Pins", *Engineering News-Record*, March 24, 1921.

† Engr. of Bridges and Bldgs., P. & R. Ry., Philadelphia, Pa.

the decrease is marked, amounting to 6% for a 150-ft. span and 20% for a 300-ft. span. A more desirable formula appears to be:

$$I = 125 - \frac{1}{8}\sqrt{2\,000\,L - L^2},$$

which follows the tests closely, increasing the impact considerably on the shorter spans and not decreasing it so much on the longer spans, when compared with the Pencoyd formula. In the writer's opinion, the formula,

$$\frac{300}{300 + \frac{L^2}{100}}$$

is not as good as the Pencoyd formula, but the formula,

$$125 - \frac{1}{8}\sqrt{2\,000\,L - L^2},$$

is better.

As all the column formulas in common use agree well with tests, within the limits allowed in good design, there seems to be no good reason for departing from the straight-line formula adopted by the American Railway Engineering Association in 1906 and 1910, namely, $16\,000 - 70\,\frac{l}{r}$, which has been widely used. The writer would place the upper limit about 13 300, which would correspond with an $\frac{l}{r}$ of 40.

A greater value should be allowed for an angle connected by lug angles than for one connected by one leg only.

A. W. Carpenter,* M. Am. Soc. C. E. (by letter).—It is to be questioned whether end floor-beams are to be preferred. They add to the expense, make back-walls less accessible, are not practicable on ends more than slightly skewed, and the writer never found any necessity for them.

Overload.—The A. R. E. A. Specifications provide, in effect, for a 75% increase in live load without more than 50% increase in the specified unit stresses in web members. Ordinarily, chord members and end-posts of trusses up to a span of 300 ft. will carry a live load increase of 60 to 75% (varying with the length of the span), with an increase of not more than 50% in the designing unit stress. In order that the interior web members may not be stressed more than 50% above the designing unit stresses, with the increase in live load which would increase the corresponding chord unit stresses 50%, the web sections must be increased slightly, and it seems well to require this as it adds comparatively little expense to the structure. The A. R. E. A. Specifications provide for the extreme case of 75% overload. By providing for an overload in interior web members equal that which the chords will carry with the same 50% increase of designing unit stress, the desired end would be attained in perhaps the most satisfactory manner.

* Asst. Valuation Engr., N. Y. C. & H. R. R. R., New York City.

Lateral Forces.—Centrifugal force taken at 6 ft. corresponds closely to the height of the center of gravity of the locomotive.

The writer sees no occasion for formulas more complicated than the straight-line formula. There is one thing in connection with column strength, however, which perhaps has not been considered and which might make a worth while subject of investigation, that is, the effect of impact on columns and compression members. The horizontal "cut-off" of the column formula for low ratios of length to radius of gyration is probably all right for static loads, but for impact it is questionable. The impact effects, as the writer conceives them, are momentary in their extremes. It requires time as well as load to cause a compression failure. The case may well be similar to that of timber beams, as noted hereafter. It may be different with a tension member. To take care of this feature the amount of impact could be made less for compression members than for tension members, although retaining the respective working unit stresses proposed. It seems likely that compression members of ordinary truss bridges are being made disproportionately heavy on this account.

Relative to tension in extreme fibers of timber, the Committee on Timber of the American Society for Testing Materials reported, in 1921, in reference to working stresses in structural timber beams, as follows:

"It is also well determined by the tests that resistance to suddenly applied loads is much greater than to slowly applied or constant loading; therefore, the condition of loading will affect the amount of allowable stress;"

and, also,

"From the above it will be seen that in order to have a rational design it is necessary to state working stresses of different amounts for different kinds of loadings and exposure of timber. As an example, for dense structural yellow pine, the maximum working stress will be 1 100 lb. per sq. in. This is for constant loading and for submerged locations where the timber is constantly wet. In locations in the weather, such as bridges, the allowable working stress for constant loading is 1 400 lb. per sq. in. Under cover, where the timber is always dry, the allowable working stress for constant loading is 1 600 lb. per sq. in."

Also,

"From the tests, it is determined that the resistance of timber is approximately proportional to the speed of loading. For constant loading, the stresses above given are proper, but for sudden loading, resulting in 100% impact, the successive loadings being far enough apart so as to allow reasonable recovery of the timber, the allowable stresses may be doubled (not to exceed 2 800 lb. per sq. in.), the stresses due to this sudden loading being those actually computed from the load with the impact. For other proportions of impact, less than 100%, the allowable working stress may be increased in a ratio equal to the percentage of impact. For dense structural yellow pine, the allowable working stresses would, therefore, be as follows:

1. For wet or submerged locations... $1\,100 + 1\,100\,I$ lb. per sq. in.
2. For exposed locations (bridges)... $1\,400 + 1\,400\,I$ " " " "
3. For constantly dry locations..... $1\,600 + 1\,600\,I$ " " " "

where I is the proportional impact stress."

These quotations from the report of the A. S. T. M. Committee, are given because the writer (who is in no way connected with that Committee)

believes it to be the most rational exposition of the subject he has seen and that it points out the proper method of specifying working stresses for timber, although the constants of the impact factors seem rather high. It will be noted that the A. S. T. M. Committee would allow, for dense structural yellow pine, 2 800 lb. per sq. in. (fiber stress in bending) for live loads with 100% impact, 2 100 lb. per sq. in. with 50% impact, but only 1 400 lb. per sq. in. for all dead load or a sustained live load without impact.

BENJAMIN W. GUPPY,* M. AM. SOC. C. E. (by letter).—A desire on the part of some members to have the Society sponsor some kind of a bridge specification has been noted; but what kind, a working specification or a general specification? A working specification is necessarily narrow. It is a definite set of rules issued by the purchaser in order to obtain the kind of a bridge he wants. It is a part of the contract. It should not be used in Court, except as evidence of what is required under the contract. When prepared by an individual, it is the embodiment of his personal predilections; when prepared by a committee, it is usually a compilation of compromises and is subject to modification by the individual user. A general specification, on the other hand, should be broad. From it the purchaser should be able to draw the necessary inspiration and information to enable him to prepare a working specification for any kind of a bridge at any location. It should furnish the information which would enable a user to determine the maximum safe carrying capacity of his structure, in so far as it is possible to do so. It should be capable of being used in Court as a guide in determining right and wrong in bridge design.

The construction of any bridge requires a minimum expenditure to secure the requisite service and security. Additional expenditures, to obtain increased strength and rigidity, to facilitate maintenance, and to minimize the effects of accidents, are optional with the purchaser. Moreover, the cost of the bridge superstructure is only one unit in the cost of the project, and should be balanced against all other items of cost, in order to obtain the total minimum expenditure. All of which should be recognized and provided for in a general specification.

Span.—What is the necessity of recommending limiting span lengths? Why not state limiting conditions instead? The maximum span for I-beams is determined by three factors: Loads, unit stresses, and section moduli. Their use is determined by these three factors plus relative economy. The purchaser should have the option of substituting built sections for I-beams, in order that the work may not be delayed, or for any other good reason, hence there is no minimum limit for plate girders. The maximum span for plate girders is determined by weights and dimensions that can be handled and shipped to the site, the question of relative economy always being considered, and this includes annual cost as well as first cost. As between riveted and pin trusses, it is a question of economy, rigidity desired, and local conditions controlling erection.

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Live Loads.—The use of the Cooper series of engines should be continued for well-known and oft-repeated reasons. The proper E value to use is a matter for individual judgment, based on past experience and an estimate of future traffic development. It is good judgment to use an E value in excess of the equivalent E values of the heaviest engines in service. A recommended percentage of excess could be stated. The proper E values for use on different classes of roads could also be recommended.

Unit Stresses and Materials.—Relative to the appropriate unit stresses to use with different grades of materials, and specifications for these materials: The relation between the various stresses should be such that a bridge will be obtained of uniform strength throughout. Would it not be practicable to write all stresses as functions of T, the axial tensile stress, and then specify the appropriate values of T for the different grades of material.

Expansion Bearings.—The object of hinged shoes is to reduce secondary stresses induced by deflection of the bridge or tipping of the masonry and to prevent the eccentric application of reaction due to deflection of the bridge. If the ends of girders are properly designed with sole-plates of the minimum size necessary to meet the specified stress for bearing of metal on metal, the displacement of the end reaction due to deflection will have no serious results. The requirement is not a necessity as has been proved by the long and successful lives of a large number of bridges not equipped with pin shoes.

The writer believes a minimum span length of 100 ft. is sufficient, except for very shallow or very heavily loaded girders, when a limiting span length of 80 ft. can be specified.

Inclined Bearings.—What valid objection can there be to allowing a short-span plate girder to slide on a plane parallel to the grade instead of on a horizontal plane. If the bridge is on a grade, something must be beveled, if there are no pin shoes. It is simplest to put the bevel in the bed-plates. Suppose the sole-plates are beveled and the bridge arrives wrong end to; it will be necessary to go to a wye or turn-table, or laboriously turn the girder at the site; or suppose, as has happened, it is discovered at the site that all the shoe-plates have been riveted up with the thick ends at the ends of the girders. On the other hand, if a casting has been incorrectly placed, it is easy to jack up and reverse it.

Any clearance diagram included in railway bridge specifications should be presented merely as a recommended diagram to be followed where it does not conflict with the law or in the absence of specific instructions. In thickly settled localities, there necessarily must be modifications in clearance often requiring legislative approval, otherwise the cost of furnishing certain facilities would be prohibitive.

Unless the floor of the bridge is planked, ties should be spaced far enough apart so that one's foot will not get wedged between them. The minimum that can safely be used is about 6 in., and 10-ft. ties are long enough, provided an occasional tie is extended and a sidewalk is built outside the guard-stick. These walks have become common in railroad yards and are being called for more and more frequently by Safety First Committees. A floor

of 12-ft. ties, without a sidewalk, does not afford a good passageway alongside a train, especially when it is coated with snow and ice.

The reduction percentages to be applied to loads for multiple-track bridges should be such that the total live load plus the impact minus the reduction, will not be less than the live load without impact. Such bridges in or near yards and terminals will be loaded on all tracks more frequently than similar bridges on the road.

Referring to unit stresses: After the correct relationship between the various stresses has been determined for each class of material, the question of the actual pounds per square inch to be used can be left to individual judgment.

Should not the combined fiber stress be limited to a tension of 16 000 lb. per sq. in., and the proper reduction be made for compression?

Referring to details of design: There will always be a difference of opinion concerning the value of certain details. In a section on this subject, particularly, a compilation of the varying clauses of different specifications will prove of value, especially if accompanied by a full discussion and explanation of each requirement.

It may be stated that local conditions exert such a great influence on the development of any design that they cannot be ignored in a general specification.

G. G. THOMAS,* ESQ. (by letter).—As the American Railway Engineering Association Specifications of 1910 appear to give economical spans, it is the intention of the writer's Company to continue their use.

The principal reasons for adhering to the 1910 Specifications are:

First.—The impact allowance called for is less for spans of 100 ft. and less than in the 1920 Specifications; this will represent a saving in tonnage, as the Company's bridges are, as a rule, plate-girder spans. The impact allowance in the 1910 Specifications appears to cover amply the actual stresses caused by impact, as far as present experience and experiment show.

Second.—The compression formula and the limiting compressive stress allowed in the newer specifications will give heavier and, therefore, more expensive trusses than the older specifications.

CHARLES EVAN FOWLER,† M. AM. SOC. C. E. (by letter).—The fact is generally admitted that the Cooper loadings are out of date, and are only being retained as a matter of expediency. The adoption of a new system that is logical and that will distribute the metal properly, as between short and long spans, was shown to be feasible, by the writer,‡ in his paper on the "Revision of the Niagara Railway Arch Bridge." The Cooper loadings could still be used as a measuring stick, and it seems presumptuous to state that railroad operating officials could not understand such a change.

The matter of impact is certainly better provided for in the elliptical curve, but for a specification presenting the ultimate, it should also have factors for variable and critical speeds.

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† Cons. Engr., New York City.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1919.

With a proper loading, full allowance for impact, secondary, and all other stresses, modern unit stresses on a basis of not less than 20 000 lb. per sq. in. for structural steel in tension, should be adopted. The writer believes engineers should go even higher and make it possible for the railways to finance the replacement of large numbers of old and light bridges. The fabricating companies should see the advantage to them and join heartily in such a movement.

The adoption of a column formula must wait on a thorough study of all the recorded test of columns. There are many engineers who believe that such a study would result in the adoption of the Gordon formula, and this may be true. It may be found, however, that a straight-line formula is practically correct; and, even if the formula adopted would prove as complicated as the secant formula, it need not frighten any one, for the unit stresses for various values of $\frac{l}{r}$ can be tabulated.

It does not matter what specifications are written or published, it is likely that each railway or each large system will continue to have its own specification. They will usually modify the specification standardized by a technical society to suit their own needs. The American Railway Engineering Association proceeded with its specification, regardless of the fact that the Society did not join in it; and the writer inquired of O. E. Selby, Jun. Am. Soc. C. E., Chairman of the A. R. E. A. Committee, about a number of things that had come up in his practice, particularly at Niagara, especially as to column formulas, unit stresses, impact, and locomotive loadings. An invitation was received to attend the Committee meeting at Pittsburgh, Pa., which meeting was held, the writer thinks, in October, 1919. The question of a column formula was discussed by F. E. Turneaure, M. Am. Soc. C. E., and about sixteen other Committee members, all of whom agreed that the logical thing to do was to take all the tests on columns, of the past thirty years, and weed out the columns that were poorly designed, poorly tested, or had failed in detail. This would leave the data on good columns, properly tested, from which to plot curves for the different end conditions, and then decide logically what column formulas to adopt, which would be likely to conform with the ideas of some of the discussors, with H. B. Seaman, M. Am. Soc. C. E., and particularly with the belief of George H. Pegram, M. Am. Soc. C. E., that, when such results are obtained, they will agree closely with the Gordon formula. The A. R. E. A. Committee did not carry out this work.

O. E. Hovey, M. Am. Soc. C. E., worked out the lines or formulas for the light and heavy sections from the report of the Special Committee on Steel Columns and Struts; and from the diagram (Fig. 15) that appears in the Niagara paper,* it may be seen that the Gordon formula, as there plotted, and the lines as plotted by Mr. Hovey, are nearly parallel for the central part of the Gordon curve, or from $\frac{40\ l}{r}$ upward; so that some Gordon formula, in the writer's opinion, will finally satisfy every one, and there is no need to worry

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 1959.

about the working out of a complicated formula, because it can be tabulated once for all.

The other major matters are unit stresses and locomotive loading. The question of unit stresses for alloy steels is referred to subsequently, but such unit stresses should certainly not be less than two-thirds of the elastic limit of any material used, or on the basis of 20 000 lb. in tension for structural steel.

It may be interesting to give some idea of the investigations being made for the Detroit long-span bridge, which must be largely of alloy steel. Nickel steel and silicon steel are being used very often, and certainly a comprehensive specification ought to cover them and the corresponding unit stresses. At Detroit, the only silicon steel offered was one with an elastic limit of 45 000 lb., and a steel with an elastic limit of 55 000 lb. is required. This was taken up with one of the best metallurgists in the United States. It is very desirable that tests should be made on the series of silico-manganese-molybdenum steels, referred to by Mr. G. W. Sargent before the American Society for Testing Materials, with carbon from 0.10 to 0.55; manganese from 0.45 to 0.75; phosphorus less than 0.04; sulphur less than 0.05; silicon from 1.75 to 2.00; and molybdenum from 0.25 to 0.50.

In the matter of locomotive loading, the A. R. E. A. Committee insisted on adhering to Cooper. The Committee had prepared a number of diagrams (two of which are published in the writer's closure of the Niagara paper*) purporting to show that the Cooper loading was close enough; E-60, however, was as much as 15% too light for the shorter spans and probably 5% too heavy for the longer spans, as compared with actual loadings. The diagrams did prove just what has been brought out in this discussion.

Referring to the matter of impact formula, it was found at Niagara that some loadings gave more than 100% on short spans. Many variations in impact were found on the 550-ft. span, as compared with other data, owing to the fact that impact is due, not only to the sudden application of the load, but also to unbalanced drivers, bad rail-joints, vibration, and all the additions that a strain-gauge, or whatever instrument is used for taking impact readings, will sum up in a total which will cover everything that is deleterious to the structure.

The writer believes that the English formula presented is the best, for a simple curve, that could be developed. C. W. Lloyd-Jones, Chief Engineer, H. E. H., The Nizam's Guaranteed State Railways, India, who came to the United States in 1921, spent some time with Professor Turneaure, and obtained a great many impact data.

Specifications should cover only points of general design and general detail, and sections covering Unit Stresses and Details of Design should be printed in a separate pamphlet for use in the designing, drafting, and templet departments of the railway and bridge companies. Likewise, sections covering the Quality of Materials and Workmanship should be printed in a separate pamphlet for the use of railway purchasing and inspection departments and the shops of the bridge companies; and there should be a third pamphlet on

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 2021 and 2023.

Inspection, Painting, and Erection. These sets of rules can then be referred to by number in the general specification.

The general specification then should cover General Designs; Artistic Features; General Data; Loads and Unusual Forces; Unit Stresses; Details of Construction; and Material and Workmanship, in two separate pamphlets; Inspection, Painting, and Erection in detail, or else in a third pamphlet; and Foundations in general, with foundation details, preferably in a separate pamphlet.

Each large railway, or at least each great system, will undoubtedly, as stated, have its own general specification, but there need be no reason for duplicating the matter suggested for printing in the three separate pamphlets.

FREDERICK E. SCHALL,* M. AM. SOC. C. E. (by letter).—The following suggestions are made with regard to railway bridge specifications: Four-track bridges may be built with four trusses, to avoid disturbing the alignment of the two inside, usually main, tracks.

Spacing of Trusses.—The minimum spacing of trusses or girders fixed at one-twentieth of the span seems to the writer not as good as the one-fifteenth fixed in the A. R. E. A. Specifications.

Weight of Materials.—With heavy rails, such as 135 lb. per yd., with tie-plates and steel guard-rails, the weight of rails and fastenings should be increased to 200 lb. per lin. ft. of track.

Impact.—The impact formula of the 1920 A. R. E. A. Specifications should meet the requirements for open-floor bridges, except for spans of 25 ft. and less. For these short spans, a reduction of the unit stress of, say, 1 000 lb. per sq. in., such as was made in some of Cooper's specifications, might be adopted to good advantage, as short spans, stringers, etc., receive the most severe usage under traffic.

For concrete ballasted floor bridges, for which a smaller impact than for open-floor bridges should be expected, a modified formula for impact should be incorporated in specifications, based on the necessary tests.

Overload.—In order to obtain a properly co-ordinated structure under a 50% overload specified, a live load 50% greater, with impact, than is desired for normal conditions, coupled with proportionately increased unit stresses, might be used in determining the sections required for the various parts.

The shearing unit stress in plate-girder and I-beam webs, if correctly stated at 10 000 lb. per sq. in., net section, seems considerably at variance with the A. R. E. A. Specifications, which allow a unit stress of 10 000 lb. per sq. in. gross section. The writer prefers the allowable unit stress based on net section, as this method tends to prevent the over-punching of web-plates.

Strength of Connections.—Connections carrying live-load stress should have a strength equal to that of the members connected, regardless of the computed stress. Connections for laterals and similar members may be proportioned for the calculated stress. Connections should be made as nearly as practical, symmetrical with the axis of the member.

* Bridge Engr., Lehigh Valley R. R., Bethlehem, Pa.

Built Tension Members.—The minimum length of stay-plates for riveted tension members should be specified.

Cover-Plates.—Flange cover-plates need not be fully developed at the theoretical ends. The requirement of the A. R. E. A. Specifications of extending the cover-plates 18 in. beyond the theoretical length seems to be more consistent.

Flange Splices.—Splices for flange members in long girders are generally necessary; the manner of such splicing should be described.

Bents and Towers.—Bents and towers should usually be united in pairs to form towers, but where single bents occur, the columns should have either hinged ends, or the columns and anchorage proportioned for bending due to horizontal forces and temperature stresses.

The yield point, in pounds per square inch for structural steel, in the writer's opinion, should be fixed at 33 000, or at least made 32 500 which would be 50% of the upper limit of the tensile strength requirement.

Class of Workmanship.—For the main sections of railway bridges, subpunched and reamed work, or drilled work, only, should be used, to assure the best results under the present bridge shop methods, where tonnage output plays such an important part.

O. H. Ammann,* M. Am. Soc. C. E. (by letter).—There appears to be no good reason why specifications cannot be written that will cover the entire field of bridge design. There is much common ground in the design of ordinary and extraordinary bridges, and the field of large bridges is practically virgin in so far as common understanding in the fundamentals of design is concerned.

Such specifications should cover the fundamental principles of sound design, load, and stress assumptions, quality of material and workmanship, as applicable to all kinds of bridges, thus allowing a wide margin for the ingenuity and judgment of the designer in the working out of the style, general arrangement, and type of details.

The true purpose of specifications is uniformity in strength. If too narrow and restrictive as to style and type of design, they tend to shackle originality and progressiveness, promote conventionality, and lack the necessary elasticity to adapt themselves to the ever-changing conditions.

Some of the more important features of specifications, on which there appears to be considerable diversity of opinion, are the live load, impact formula, and unit stresses, more particularly the compression formula.

Live Load.—The Cooper engine loading has become a standard by which railroad bridges can be easily gauged as to carrying capacity, and should not be lightly abandoned, unless another engine loading is produced that is more truly representative of the great variety of present and future locomotives.

Compared with any equivalent uniform load, which, after all, must be based on actual weights, any engine loading possesses the not insignificant advantage of conveying to the mind a truer picture of the actual load.

* Cons. Engr., New York City.

In the Cooper loading both the engine concentrations and the uniform train load behind the engines can be adapted, by simple proportion, not necessarily the same for both, to the actual increase in weight of these train units, and the loading has thus sufficient flexibility for all practical purposes.

Impact.—The new impact formula of the American Railway Engineering Association, is an improvement on the old formula, because it results in a justified economy of metal for spans longer than about 100 ft.; but it is less simple and retains the defect of the old formula of giving impact values that are too low for single-load concentrations and short lengths of load, up to about 25 ft.

Mr. Seaman's formula* and the simpler one proposed by Maj. Mount are better in this respect and, for all greater span lengths, give sections (the only proper criterion for comparison) which differ by not more than 5% from those obtained by the proposed formula.

On account of its simplicity, the writer considers Maj. Mount's formula, $\left(\frac{120}{90 + L}\right)$, or a slight modification thereof, much to be preferred to any of the other purely empirical formulas.

All the previously mentioned formulas, if used in connection with a basic unit stress of 16 000 lb. per sq. in., result in a waste of metal in long spans of, say, more than 500 ft. This is not due so much to the impact formula as to the low unit stress, but if a higher unit stress were allowed, the formulas would give too low values for short spans; in other words, the impact formulas and the unit stresses are not properly correlated for all span lengths.

The simple empirical formula, $\frac{200}{100 + L}$, in combination with a unit stress of 20 000 lb. per sq. in., would give good results for all span lengths.

Whichever of the aforementioned formulas may be adopted, the writer would recommend that the length, L, should be taken to mean the aggregate loaded length on all tracks. This will result in a justified economy in bridges with more than one track. Although no comparative test results are available, it is only reasonable to expect that the impact percentage decreases with increasing number of tracks.

Further, on account of the "cushioning" effect of the ballast, it is reasonable to expect, and the English tests, barring erratic results, prove, that the impact in ballasted bridges is considerably less than in open-floor bridges. The writer, therefore, would recommend, pending further investigation, that ballasted bridges be proportioned for only a fraction, say, 75% of the impact allowance in open-floor bridges.

The semi-empirical formula,† of Gustav Lindenthal, M. Am. Soc. C. E.

$$i = \frac{l}{l + d} \cdot \frac{1\,200 + \frac{a}{n}}{600 + 4a},$$

whatever justification its derivation may have, is the only one which takes

* See p. 595.

† *Engineering News*, August 1, 1912.

all the factors mentioned automatically into consideration. In combination with the high basic unit stress of 20 000 lb. per sq. in., it gives reasonable results in all cases.

For single-track bridges with open-tie floor, and spans up to about 500 ft., it results in steel weights that agree fairly well with those of the other formulas. In longer spans, in bridges with ballasted floors, and in bridges with more than one track, it effects a justified saving.

Unit Stresses and Column Formula.—The writer does not favor the use of low unit stresses. He would rather make liberal assumptions for the external forces and possible combinations of such forces, and let the maximum stresses under extreme conditions reach high values (close to the elastic limit in tension members and to two-thirds of the buckling strength of compression members), provided that design and details are excellent in every respect.

It may be apprehended that the indiscriminate use of the proposed low unit stresses for compression, in particular, will result in unnecessary waste of metal. There appears to be no justification for such low unit stresses for properly designed compression members of small values of $\frac{l}{r}$. These low stresses will encourage poor design under cover of a safe margin, rather than result in greater safety.

In the writer's opinion, the old American Railway Engineering Association formula, $16\,000 - 70\,\frac{l}{r}$, in connection with the unit of 16 000 lb. per sq. in. for tension, gives a safe stress for properly designed columns. The specifications should prevent poor design by prescribing suitable reductions in the permissible stress in cases where thin unstiffened webs, flimsy latticing, such as flat lattice-bars, wide unsupported flanges, or composite parts of the section without direct lateral support, are used.

The Rankine formula is unnecessarily complicated. Its apparent scientific form is not justified, because the law of proportionality, on which its derivation is based, does not apply to the process of buckling of short columns, as the fiber stresses exceed that limit long before actual buckling takes place.

For long columns, with $\frac{l}{r}$ greater than the practical limits, Euler's formula is the only correct theoretical formula. It would be applicable also to shorter columns if the modulus of elasticity was introduced as a variable value for stresses greater than the limit of proportionality. In that form, it proves the fact that the buckling strength of a very short column $\left(\frac{l}{r} = 0\right)$ is identical with the yield point, that is, the stress for which the modulus of elasticity in steel becomes zero.

For intermediate values of $\frac{l}{r}$, which comprise the entire range of practical values, a purely empirical formula and, as the simplest form of such, the straight-line formula appears to be as good as any other.

J. B. HUNLEY,* ESQ. (by letter).—Materials and workmanship have been highly developed, and little difference of opinion can exist as to these requirements in specifications; but there are various ideas as to the type of live load and impact formulas which should be used, and the allowable intensity of working stresses, particularly for columns or struts.

Live Load.—It is admitted that the Cooper loading does not closely conform to any type of engine actually in use, but nothing better has hitherto been devised, and its use has become so general, not only for the design of new railroad bridges, but particularly for the rating of existing bridges and engines, that it is doubtful whether it should be abandoned. It seems to be quite as representative of the modern heavy engine as it ever was of the older and lighter engine, and it is now so firmly established that the time for changing it has probably passed, unless there is in the future some radical change in engine or car design.

Impact Formula.—It is possible that a formula should be used providing more than 100% impact for short spans. Some recent experiments indicate that such high impact stresses exist, but, unfortunately, few tests have been made with American equipment. It is quite probable that, even under average conditions of track and equipment, the impact stress on short spans exceeds the live load; and tests in sufficient number should be made in order to arrive at some correct formula.

It has been contended that the special loading for short spans, consisting of two or more heavy concentrations, safely provides for impact stresses in excess of 100%, but this may be questioned. It is a well-known fact that individual axle loads of locomotives may, and frequently do, exceed the average or assumed axle load by as much as 40 or 50%, with improperly adjusted spring hangers, and the specified concentrated loads can do little more than provide for this contingency.

Column Formula.—The straight-line formula has little to recommend it, except its simplicity and the fact that it does fit, with some accuracy, the results of tests. The second term means little, mathematically, but is merely an approximation of the decrease in strength resulting from an increase in length. It is theoretically incorrect, as its values form a straight line, whereas the actual values must approximate a reverse curve. The values representing the strength of a steel column would not continue horizontally along the line, representing the yield point of the material, to an intersection with the Euler curve, but would drop off between points representing the actual and apparent yield points before it intersected the Euler curve, even if no bending was introduced before the critical load was reached. This results in a reversed curve which, for the actual column, is accentuated by its inherent defects, eccentricities, etc.

Other formulas, more rational, giving accurate results and but little more difficult of application, are available. The mere computation of the permissible working stress in a column is a small part of the careful design or thorough investigation of such members, and if the straight-line formula has no particular merit other than ease of application, it is time to discard it.

* Engr., Bridges and Structures, C. C. C. & St. L. Ry., Cincinnati, Ohio.

The parabolic formula, $p = 12\,500 - 0.25\left(\frac{l}{r}\right)^2$, has much in its favor and, although it is purely empirical, it fits closely the plotted results of tests and is about as easy to apply as the straight-line formula for all ordinary values of $\frac{l}{r}$ less than 160 or 170, where, as often written, $13\,000 - 0.25\left(\frac{l}{r}\right)^2$, it becomes practically tangent to the Euler curve. It would appear that 13 000 could be used safely for the first term of this formula, instead of 12 500. When this value is used and the results are multiplied by 2.5, it will be seen that the curve falls well within, or even below, the averages of tests of pin-end columns.

The Rankine formula, $p = \frac{16\,000}{1 + \frac{l^2}{13\,500\,r^2}}$, if multiplied by the factor 2.5, representing a yield point of 40 000 lb., becomes tangent to the Euler curve, $\frac{16\,E}{\left(\frac{l}{r}\right)^2}$, when the ratio of l to r is about 300. It gives too high values of p, for values of $\frac{l}{r}$ from 0 to 75, and too low values between 90 and 250. This has been the principal objection to this formula. The factor in the second term of the denominator cannot be adjusted to make the curve fit the tests for all values of $\frac{l}{r}$. The curve of this formula, as shown by Mr. Seaman,* is made to fit the average results of tests on flat-end columns, by multiplying the formula by the factor 2.25. This does not make the curve tangent to the Euler curve, and it assumes the yield point of the material as 36 000 lb., when it is probably nearer 40 000 lb.

It seems to the writer that the secant formula is well worth consideration. It is theoretically correct for all lengths of the ideal column centrally or eccentrically loaded, and by the assumption of one factor will properly provide for any inherent eccentricities and defects of the column. With very small values of $\frac{l}{r}$, the working stress approaches the allowable fiber stress and, with increasing values of $\frac{l}{r}$, the formula approaches the Euler formula, which is known to be correct for long slender columns.

Its curve follows closely that of the parabolic formula, for columns centrally loaded, but contains a factor for eccentrically loaded columns; and, for such columns, the term, c, representing one-half the width, takes into account, to some extent, the shape of the section.

The formula as usually written is:

$$p = \frac{f}{1 + \frac{e\,c}{r^2} \times \sec.\ \frac{l}{2}\sqrt{\frac{P}{E\,I}}} \qquad \text{(IX)}$$

* See p. 597.

which may be written:

$$p = \frac{f}{1 + \frac{e\,c}{r^2} \times \sec. \frac{l}{2r}\sqrt{\frac{P}{E}}} \quad \text{.....(X)}$$

in which,

P = load on column;
A = area of column section;
$p = \frac{P}{A}$, average stress;
f = working stress;
e = eccentricity of application of load;
c = distance from neutral axis to outer fiber.

An examination of Formula (X) indicates that when $\frac{l}{r} = 0, p = f$, where there is no eccentricity. As $\frac{l}{r}$ increases, $\frac{l}{2r}\sqrt{\frac{P}{E}}$ approaches the value, $\frac{\pi}{2}$, and the formula becomes the Euler curve, when there is no eccentricity. It also indicates that p does not increase in the same proportion as f.

In using the ordinary column formula for an eccentrically loaded column, the average stress is, $p = \frac{f}{1 + \frac{e\,c}{r^2}}$, whereas, in Formula (X), the term, $\frac{e\,c}{r^2}$, is multiplied by sec. $\frac{l}{2r}\sqrt{\frac{P}{E}}$.

This formula is not widely used, and has apparently not received the consideration it deserves, perhaps for the reason that it is more difficult to apply than many other formulas. By expanding the secant term and using only the first two terms of the series, the formula may be simplified, and becomes:

$$p = \frac{b}{2} \pm \sqrt{\left(\frac{b}{2}\right)^2 - C} \quad \text{.....(XI)}$$

in which,

$$b = \frac{8\,E}{K^2}(1 + a) + f$$

$$C = \frac{8\,E}{K^2} \times f$$

$$K = \frac{l}{r}$$

a = some value, including $\frac{e\,c}{r^2}$, where known eccentricity occurs, and a factor to provide for unknown eccentricity and defects, and for the condition at end of column, whether pivoted, hinged, or fixed.

It may be said that the formula is now only an empirical one, with a value, a, selected to form the desired curve. Although it is not so easily recognized, it still contains the bending factor. When the third term of the expansion

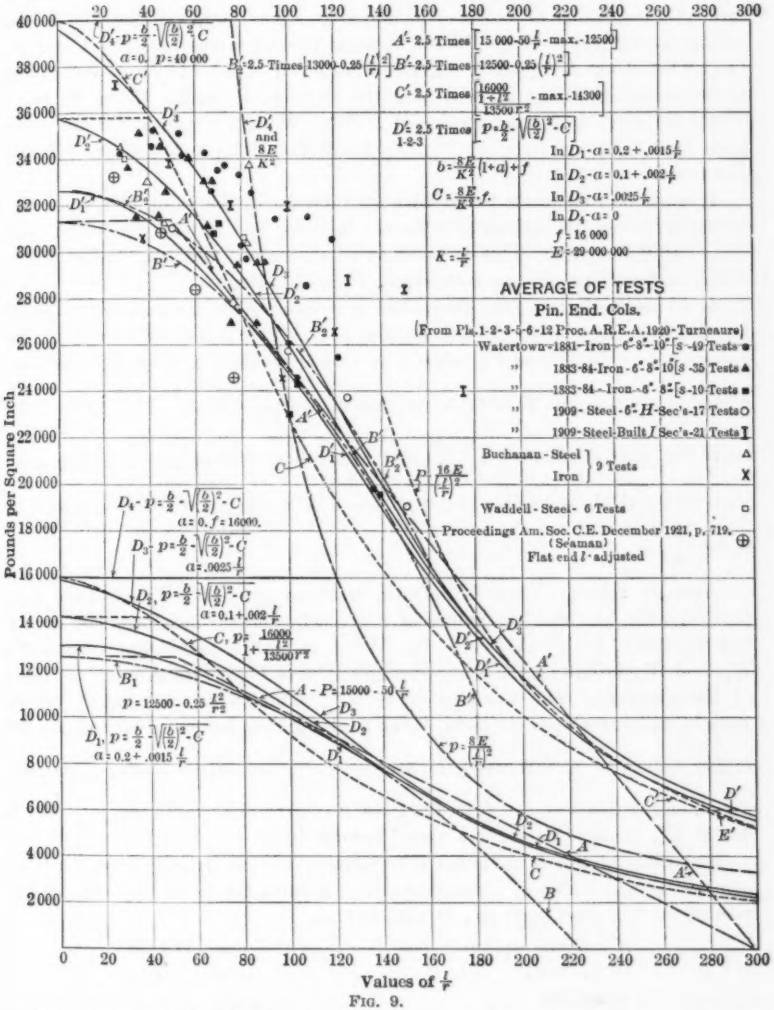


FIG. 9.

series is included, slightly different values of p are obtained, but the equation becomes more complicated and its inclusion is scarcely justified for any practical purpose; but with tabulated values of $\frac{8E}{K^2}$, $\frac{8E}{K^2} \times f$, etc., the formula in this suggested form is not forbidding enough to condemn it.

The values of p (with $E = 29\,000\,000$ and $f = 16\,000$) and with values of $a = (0.2 + 0.0015\ K)$, $(0.1 + 0.002\ K)$, and $(0.0025\ K)$, have been plotted on Fig. 9 (Curves D_1, D_2, D_3), with three other formulas. Curve D_1 ($a = 0.2 + 0.0015\ K$), practically coincides with the parabolic formula through a long range of values of K, and gives a maximum value of $p = 13\,000$, corresponding to the parabolic and straight line maximum values. Curve D_2 ($a = 0.1 + 0.002\ K$) gives a maximum value of about 14 000, corresponding to the maximum value allowed with the Rankine formula. Curve D_3 ($a = 0.0025\ K$) gives a maximum value of 16 000. When the values of p are multiplied by 2.5, the curves, D'_1, D'_2, D'_3, touch the Euler curve, when K equals 200 to 250.

It must be remembered that with an increase in the value of f from, say, 16 000 to 40 000, the values of p do not increase in the same proportion with the same value of a. This is peculiar to the secant formula, and is due to different functions of p in both sides of the equation.

It is interesting to note that when a is assumed to be zero, which would be the case for ideal pivoted columns, and $f = 16\,000$ (Curve D_4), the values of $p = 16\,000$ until a value of $K = 120$ is reached, where the curve becomes $\frac{8\,E}{K^2}$, or, if $f = 40\,000$, the curve becomes $\frac{8\,E}{K^2}$ with $K = 76$. The Euler curve for pivoted columns is $\frac{10\,E}{K^2}$, and if the other terms of the expansion series were retained, the curve for larger values of K would more nearly approach or equal $\frac{10\,E}{K^2}$.

Average values of the Watertown tests on pin-end columns have been plotted, with 2.5 times the values of p given by the various formulas, including the curve, $B'_2 = (13\,000 - 0.25\ K^2)$. It will be noted that Curves D'_1, B'_1, and B'_2, fall along the lower averages; Curve D'_2 passes through the mean of the averages. The Rankine curve, C'_1, exhibits its well known defect, namely, cutting across the field, being high for low values of K and low for higher values of K. When known eccentricity occurs, the term, $\frac{e\,c}{r^2}$, is computed, and if this was, say, 0.25, the term, a (Curve D_1), would become $(0.45 + 0.0015\ K)$, or, for Curve D_3, a would become $(0.25 + 0.0025\ K)$.

The value of this formula for investigating an eccentrically loaded column can best be illustrated by a concrete case. Assume the usual upper chord section of the box type, open on one side, having,

$A = 66.8$ sq. in.	$c = 11.8$ in.
$I = 5\,895$	$e = 2.0$ in.
$r = 9.4$	$\frac{e\,c}{r^2} = 0.267$

The safe load, $P = p\ A$, for the column will be computed by the formula, $12\,500 - 25\left(\frac{l}{r}\right)^2$, and the modified secant formula, using $a = 0.2 + 0.0015\ K$

(Curve D_1), and for values of $K = 50$, 100, and 150. It will be noted from Fig. 9 that the two curves are almost identical for this range of K. The computed values of P are as given in Table 1.

TABLE 1.

Parabolic Formula.				Secant Formula.			
$K = \frac{l}{r}$	$p_1 = 12\,500 - 0.25\left(\frac{l}{r}\right)^2$	$p = \frac{p_1}{1 + 0.267}$	$P_1 = p\,A$	$1 + a$	p	$P_2 = p\,A$	$\frac{P_2}{P_1}$
50	11 875	9 370	625 900	1.542	9 960	665 300	106.3 %
100	10 000	7 890	527 100	1.617	8 190	547 100	103.8 %
150	6 875	5 430	362 700	1.792	5 740	383 400	104.4 %

The secant formula, either in the original or in some modified form, is so rational and has so many good qualities that any discussion of column formulas would not be complete without including it, and the writer can recommend it as an interesting study that, in time, may lead to its more general use, which he believes it deserves.

Table 2 of values of $\frac{8\,E}{K^2}$, $\frac{8\,E}{K^2} \times f$, etc., is included for the convenience of any one desiring to use the formula in this modified form. The values are shown

TABLE 2.—Values of Terms in Modified Secant Formula

$E = 29\,000\,000$ $\qquad$ $b = \frac{8\,E}{K^2}(1 + a) + f$

$f = 16\,000$ lb. $\qquad$ $C = \frac{8\,E}{K^2} \times f$

$p = \frac{b}{2} \pm \sqrt{\left(\frac{b}{2}\right)^2 - C}$

$\frac{l}{r} = K$	K^2	$\frac{4\,E}{K^2}$	Diff. / 10	$a = 0.2 + 0.0015\frac{l}{r}$	$a = 0.1 + 0.002\frac{l}{r}$	$a = 0.0025\frac{l}{r}$	$C = \frac{8\,E}{K^2} \times f$	Diff. / 10
10	100	1 160 000	87 000	0.215	0.120	0.025	37 120 000 000	2 784 000 000
20	400	290 000	16 111	0.230	0.140	0.050	9 280 000 000	515 555 000
30	900	128 889	5 639	0.245	0.160	0.075	4 124 448 000	180 445 000
40	1 600	72 500	2 610	0.260	0.180	0.100	2 320 000 000	83 520 000
50	2 500	46 400	1 418	0.275	0.200	0.125	1 484 800 000	45 370 000
60	3 600	32 222	855	0.290	0.220	0.150	1 031 104 000	27 357 000
70	4 900	23 673	555	0.305	0.240	0.175	757 536 000	17 754 000
80	6 400	18 125	380	0.320	0.260	0.200	580 000 000	12 173 000
90	8 100	14 321	272	0.335	0.280	0.225	458 272 000	8 707 000
100	10 000	11 600	201	0.350	0.300	0.250	371 200 000	6 442 000
110	12 100	9 587	153	0.365	0.320	0.275	306 784 000	4 899 000
120	14 400	8 056	119	0.380	0.340	0.300	257 792 000	3 814 000
130	16 900	6 864	95	0.395	0.360	0.325	219 648 000	3 027 000
140	19 600	5 918	76	0.410	0.380	0.350	189 376 000	2 442 000
150	22 500	5 155	62	0.425	0.400	0.375	164 960 000	1 997 000
160	25 600	4 531	52	0.440	0.420	0.400	144 992 000	1 654 000
170	28 900	4 014	43	0.455	0.440	0.425	128 448 000	1 389 000
180	32 400	3 580	37	0.470	0.460	0.450	114 560 000	1 174 000
190	36 100	3 213	31	0.485	0.480	0.475	102 816 000	1 002 000
200	40 000	2 900	25	0.500	0.500	0.500	92 800 000	805 000
220	48 400	2 397	19	0.530	0.540	0.550	76 704 000	613 000
240	57 600	2 014	15	0.560	0.580	0.600	64 448 000	477 000
260	67 600	1 716	12	0.590	0.620	0.650	54 912 000	378 000
280	78 400	1 480	10	0.620	0.660	0.700	47 360 000	306 000
300	90 000	1 289		0.650	0.700	0.750	41 248 000	

Note.—Differences are divided by 20 for $\frac{l}{r}$ from 200 to 300.

for values of K, varying by 10; for any intermediate ratio, the value can be obtained by interpolating, with accurate results.

OTIS E. HOVEY,* M. AM. SOC. C. E. (by letter).—It is perfectly proper for the engineer of a railroad to describe in detail just the kind of bridges he may wish to use on his own road; but the writer feels that the Society should attempt to prepare a genuine general specification, laying down such requirements as are vital to good design, suitable materials, proper workmanship and construction, or leave the field to those who wish to express their personal preferences for certain details through the medium of their specifications.

A thoughtful study of the regulations prescribed by law in France might assist one to approach the problem in a manner quite different from that of the Special Committee on Specifications for Bridge Design and Construction. These regulations are of a general character, and a brief paragraph in the second article of the Act of August 29, 1891, states that "these formulas are given as a matter of information only and do not limit in any way the initiative of the engineers, who may employ any method which they may deem expedient", and this refers to as vital a matter as unit stresses.

In this is the nucleus of the whole matter. The Society should endeavor to stimulate and not hamper the development of the best principles of bridge design. The Society should encourage thought, originality, and initiative in its members; but, at the same time, it should be sure to place before them the broad general principles to be followed, and epitomize the experience of the many thoughtful and competent engineers who have given their best efforts to the development of good practice in bridge design and construction.

The writer had intended to limit his discussion to the general principles previously mentioned, but has been requested to present the following method of designing the latticing of compression members. The method is not new, having been used for many years, but may be of value, as it is developed with special reference to the Rankine column formula.

The Rankine column formula, which, in form, is easily derived from the Euler equation, may be stated as follows:

$$\frac{P}{A} = \frac{S}{1 + \frac{l^2}{c\,r^2}} \qquad \text{(XII)}$$

in which,

P = the total axial load on the column,
A = its area,
S = the permissible maximum fiber stress,
l = its length,
r = its radius of gyration, and
c = an experimental constant.

For a short column, $\frac{P}{A} = S$, so it is evident that the expression, $\frac{l^2}{c\,r^2}$, is introduced to provide for bending stresses in a column of material length.

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By solving for S,

$$S = \frac{P}{A} + \frac{P}{A} \times \frac{l^2}{c\,r^2}$$

and the portion, $\frac{P}{A} \times \frac{l^2}{c\,r^2}$, is the allowance for bending expressed in unit stress, or one may write:

$$f = \frac{P}{A} \times \frac{l^2}{c\,r^2} \quad \text{.....(XIII)}$$

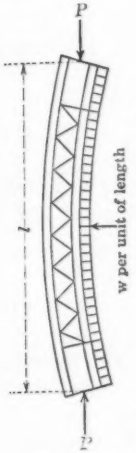


FIG. 10.

A bent column with round ends will take the general form shown in Fig. 10. In order to arrive at a method of calculating lattice bars, it may be assumed as, at least, an approximation, that the forces acting to bend the column under axial load may be replaced by a horizontal unit load of w acting uniformly over the length of the column.

Referring to Fig. 11, the bending moment at the middle of the column will then be:

$$M = \frac{w\,l^2}{8}, \text{ but } R = \frac{w\,l}{2}, \text{ hence } M = \frac{R\,l}{4},$$

and we may write:

$$M = \frac{R\,l}{4} = \frac{f\,I}{y}$$

in which f is the extreme fiber stress due to bending, I is the moment of inertia of the column section, and y is the distance of the extreme fiber from the neutral axis. Replacing I by the equivalent value, $A\,r^2$,

$$\frac{R\,l}{4} = \frac{f\,A\,r^2}{y}, \text{ but } f = \frac{P}{A} \times \frac{l^2}{c\,r^2},$$

hence,

$$\frac{R\,l}{4} = \frac{P\,l^2}{c\,y},$$

and,

$$R = \frac{4\,P\,l}{c\,y} \quad \text{.....(XIV)}$$



FIG. 11.

If Equation (XII) takes the form,

$$\frac{P}{A} = \frac{S}{1 + \frac{l^2}{13\,500\,r^2}}$$

Equation (XIV) becomes

$$R = \frac{4\,P\,l}{13\,500\,y} \quad \text{.....(XV)}$$

R is the calculated shear in the end lattice-bars of the column. If the bars make an angle of θ with the axis of the column and the column is latticed on two sides, the stress in each lattice-bar becomes

$$S_1 = \frac{R \text{ cosec. } \theta}{2} \text{ for single lacing;}$$

and,

$$S_2 = \frac{R \text{ cosec. } \theta}{4} \text{ for double lacing.}$$

The compression formula of the American Railway Engineering Association Specifications of 1910 is $S = 16\,000 - 70\,\frac{l}{r}$. Proceeding in a similar manner, the allowance for flexure corresponding to the term, $70\,\frac{l}{r}$, reduced to shear at right angles to the column is found to be:

$$R = 280\,\frac{A\,r}{y} \quad \text{.....(XVI)}$$

The values of S_1 and S_2 are in the same form as given for the Rankine formula.

The column formula given in the A. R. E. A. Specifications of 1920 is $S = 15\,000 - 50\,\frac{l}{r}$. From this is obtained:

$$R = 200\,\frac{A\,r}{y} \quad \text{.....(XVII)}$$

It will be interesting to compare the values of R obtained from Equations (XV), (XVI), and (XVII), with an actual column. For this purpose, assume a column made up of two channels, 15 in. by 40 lb. per ft. The value of r is 5.43 in. and $r^2 = 29.48$. Assume column lengths from 24 to 40 ft., varying by intervals of 2 ft.; calculate the capacity of the column for each length, and find values of R for each length. The results are given in Table 3.

TABLE 3.

$$S = \frac{16\,000}{1 + \frac{l^2}{13\,500\,r^2}}; \quad R = \frac{4\,P\,l}{13\,500\,y}$$

Length, in feet.	Length, in inches.	$\frac{l}{r}$	S= unit stress allowed.	Area, in square inches,	S= capacity of column, in pounds.	R= shear, in pounds.	Total stress in lattice-bars, in pounds at 45°	Percentage of column stress.
24	288	53.0	13 238	23.52	311 400	3 540	5 010	1.61
26	312	57.4	12 860	"	302 500	3 730	5 280	1.75
28	336	61.9	12 455	"	292 900	3 890	5 500	1.88
30	360	66.3	12 069	"	283 900	4 040	5 710	2.01
32	384	70.7	11 675	"	274 600	4 170	5 900	2.15
34	408	75.1	11 282	"	265 390	4 280	6 050	2.28
36	432	79.5	10 893	"	256 200	4 370	6 180	2.41
38	456	84.0	10 508	"	247 200	4 450	6 290	2.54
40	480	88.4	10 133	"	238 400	4 520	6 390	2.68

From the formula, $S = 16\,000 - 70\,\frac{l}{r}$, $R = 280\,\frac{A\ r}{y}$. Substituting the values from the column previously mentioned, R is found to be equal to 4 470

lb. and the lattice-bar stress at 45° is 6 740 lb., and this value is constant for all column lengths.

From the formula, $S = 15\,000 - 50\,\frac{l}{r}$, $R = 200\,\frac{A\,r}{y}$, hence $R = 3\,410$ lb. and the lattice-bar stress at 45° is 4 820 lb. This is also constant for all column lengths.

The writer feels that it is proper and desirable for the Society to prepare a broad general specification for bridges. Matters of minute detail should be avoided, leaving users of the specifications free to express their own preferences. He believes that such a general specification will be favorably received by the Profession.

There has been quite an extended discussion of various impact formulas, and it may be useful to comment briefly on some phases of the history of their development and use. During the decade from 1880 to 1890, many American engineers began to use the methods of proportioning members that had been developed by Launhardt, Weyrauch, and others, based on the fatigue tests of Wöhler and Bauschinger in Germany and those of Sir William Fairbairn and the late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., in England.

In 1885, the late Joseph M. Wilson, M. Am. Soc. C. E., Engineer of Bridges and Buildings of the Pennsylvania Railroad, issued a specification for wrought-iron bridges in which it was stated that members subject to one kind of stress, all tension or all compression, should be designed for working stresses, a, determined from the formula:

$$a = u\left(1 + \frac{\text{minimum stress in the piece}}{\text{maximum stress in the piece}}\right)$$

Members subject to reversing stresses were designed for working stresses:

$$a = u\left(1 - \frac{\text{maximum stress of lesser kind}}{2 \times \text{maximum stress of greater kind}}\right)$$

The value, u, depended on the kind of iron used, and was 7 000 lb. per sq. in. for single-rolled iron plates and shapes in tension, and 7 500 lb. per sq. in. for double-rolled iron in tension, for links and rods.

Although this method of designing produced a satisfactory distribution of material, it was cumbersome, as each member and its details and connections, including rivets, required the use of a working stress, a, peculiar to the member. As a result of this fact, various attempts were soon made to apply corrections to the calculated stresses, so that by the use of a constant unit stress basis per square inch, the same, or a closely similar, distribution of material would be accomplished. These adjustments of the calculated stresses were called "impacts", but the writer believes they had little or no reference to impacts due to the passage of moving loads. The formula* developed by the late Fred Thompson, M. Am. Soc. C. E., and first published in the specifications of the Southern Railway Company in 1894, was in the familiar form,

$$I = S\,\frac{300}{L + 300}$$

* See discussion by J. B. French, M. Am. Soc. C. E., *Transactions*, Vol. LXXV (1912), pp. 363 to 370.

This convenient formula was at once adopted by the late C. C. Schneider, Past-President, Am. Soc. C. E., as simpler and more satisfactory than his own, and has been generally used in the United States, Canada, and India for many years.

The writer feels that it was unfortunate that the stress adjustment was called "impact", and this term may have misled some users of the method not fully advised of the history of its development.

In 1908, the writer prepared a specification for bridges based on the fatigue method of design, and found that by making the value of u, in the Pennsylvania Railroad formula of 1885, equal to about 9 000 lb. per sq. in. for structural steel, the areas of the main members of single-track bridges, up to 200 ft. span, became almost identical with those of the same spans designed by the use of the "impact" formula, previously given, using unit stresses based on 17 000 lb. per sq. in. This fact tends to confirm his belief that, for practical purposes, the use of the so-called "impact" formula,

$$I = S \frac{300}{L + 300}$$

is mainly a convenient method of distributing the material in a bridge in order to conform practically to the fatigue method of design.

The real test of good design is the behavior of bridges under heavy traffic. When long service and low maintenance costs are considered, the writer believes that bridges designed by the fatigue method have been more satisfactory, on the whole, than those designed by most other methods. In view of the recent fatigue experiments, and studies now being conducted, he feels that this subject should be carefully considered in connection with the many observations of impact due to passing loads now available, before the definite adoption of a new impact formula.

J. W. Spiller,* Esq. (by letter).—A Sub-Committee of the British Engineering Standards Association, of which the writer is a member, is, at present, drafting a specification similar to the Tentative Specifications. In the first place, this Sub-Committee prepared a specification covering "Materials and Workmanship", and then divided into panels to deal with: (*a*) Loads; (*b*) Stress Units; and (*c*) Details of Design and Construction. The writer was placed on the panel dealing with "Details of Design and Construction".

The writer will confine his comments to questions of material, workmanship, and the details of design and construction.

Referring to the "nominal diameter" of a rivet: In Great Britain, there is a difference of opinion, particularly between bridge builders and shipbuilders, as to the exact point on the tapered shank where the diameter should be measured. It might be well, therefore, to define clearly the "nominal diameter".

It has also been stipulated that "connections shall have a strength at least equal to that of the members connected, regardless of the computed strength". This appears to require some qualification in the case of members which are in compression where the strength of the member will depend on the value

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of the ratio, $\frac{l}{r}$, whereas the strength of the connections will b independent of this ratio.

Limiting Thickness of Metal.—Some differentiation might be made perhaps between plates, etc., which are accessible for painting on one side and those which are accessible for painting on both sides.

Pitch of Rivets.—The maximum pitch proposed seems to be too great to insure that no corrosion will take place between the parts connected.

Splices.—The gross area of the splice material proposed, namely, 50% of the gross area of the smaller member, does not appear to the writer to be sufficient.

Stiffeners.—The Tentative Specifications do not permit the use of T-beams which make a suitable and economical form of stiffener for small spans.

Camber.—The writer is inclined to think that it is better practice to make the camber equal to the deflection produced by both dead and live load. If, then, one-half the camber due to live load is taken out by notching the sleepers, the level of the rails at the span center will be lower than that at the support by an amount equal to one-half the deflection due to live load when the bridge is loaded and will be higher by a similar amount when the bridge is not loaded.

Punching.—The limits of thickness for punching appear to be on the high side for first-class workmanship.

Web-Plates.—The large clearance allowed between ends of web-plates, namely, $\frac{3}{8}$ in., does not appear to be quite consistent with first-class workmanship.

No reference is made in the specification to the supply of service bolts which should be provided by the bridge fabricator, whether or not his contract includes erection at site.

The writer should like to add that the foregoing comments are not advanced in a spirit of criticism. The Special Committee on Specifications for Bridge Design and Construction has submitted a valuable specification and it is with considerable diffidence that the writer has ventured to draw attention to a few small matters of detail which appear to differ from what would be considered first-class practice in Great Britain.

J. B. French,* M. Am. Soc. C. E.—Many matters of detail are specified rigidly, which, under certain conditions, cannot be complied with and which, in other cases, should preferably be left dependent on the conditions of the particular case. As an example of a requirement impossible of general observance, the clearance diagram may be cited. The clearances specified, which are intended to clear a brakeman on the top or side of a box car, are desirable where practicable, but in grade separation work in large cities, such clearances would involve unwarranted expense. An alternative diagram should be given to cover the latter conditions. Again, the one live load rigidly specified is an example of a matter that should be left more flexible for the reason that many railroad structures, particularly in and around cities, are used almost exclusively for passenger traffic, which is much lighter than the E-60 loading

* Cons. Engr., New York City.

specified. If that loading is retained, the words "unless otherwise specified" should be inserted.

In regard to impact, the speaker is less concerned with the exact form of the formula used to establish the relation of impact to length of span than he is with the principle that the ratio of live to dead load shall be taken into account in its application. Some years ago, he was called on to design a large number of railroad bridges within city limits where solid floors were necessary and where fully ballasted tracks and concrete slab floors were considered desirable. In the preliminary studies, it soon became evident that, unless the usual rules of design as regards unit stresses and impact were modified, this extra dead load would materially increase the weight of the main structures and increase their cost when compared with the old type of open-deck bridges. Considering the cushioning effect of the ballast and the added stiffness and solidity contributed by the concrete, which was thoroughly bonded to the steelwork, it seemed justifiable to modify the specifications by taking these facts into consideration and thereby reduce the weight of the steelwork. The following clause, therefore, was inserted in the specifications:

"In the case of main girders or trusses, and their supports, carrying fully ballasted track, one-half the dead load stress shall be subtracted from the full live load stress, before applying the impact formula."

It seems pertinent to state that the bridges designed in this manner have now been carrying very heavy freight traffic for more than 15 years and, at present, show no signs of weakness. The speaker would continue to use such a rule of design under similar conditions to-day.

The speaker believes that the unit stresses to be used in structural design should be determined by the known physical properties of the material rather than by the character of its use, and that variations in the conditions of use should be provided for logically by corresponding variations in the assumptions of the external forces to be resisted.

The unit stresses in the Tentative Specifications were based on 16 000 lb. per sq. in. for axial tension. This value has in its favor the fact that in recent years it has been adopted, not only in specifications for railroad bridges, but also in specifications for highway bridges and for the structural steelwork of buildings. However, it seems to be too low for all these purposes and the adoption of 20 000 lb. per sq. in. instead of 16 000 lb. would be justified by the known physical properties of structural steel.

A conclusive argument in favor of higher unit stresses is contained in the specifications themselves, in that an overload of 50% is provided for, and the rule, to retain in service existing bridges the unit stresses of which do not exceed 24 000 lb. per sq. in. in tension, is regarded as safe and conservative.

Why not, therefore, increase the live load to E-75, and base unit stresses on a tension of 20 000 lb. per sq. in., or to E-90, and base unit stresses on a tension of 24 000 lb. per sq. in.

The use of structural steel and concrete in effective combination is not referred to. On account of the effectiveness of concrete as a protective covering for steel work, its availability to fill spaces and exclude water, its fire-proof

qualities, and its strength and permanence, its use as a secondary material in the construction of steel bridges has grown steadily in favor and has assumed proportions warranting its consideration in specifications of this kind.

The rules for the design of reinforced concrete have been thoroughly discussed and developed, but where steel is the predominating material and concrete is used to protect and reinforce it, comparatively little has been done to develop or encourage rational design.

It seems obvious that, where concrete is used in combination with structural steel, the more effectively the two are bonded, the more effectively the composite structure will serve its purpose and, if the materials are effectively bonded, the concrete will take its share of the compressive stresses and relieve the steelwork to which it is bonded. Consideration of these facts justify material reductions in the sections of steel parts embedded in concrete and carrying compressive stresses induced after the concrete has set, and substantial economy can be accomplished by corresponding rules of design. The speaker has done this by the use of unsymmetrical beams and girders embedded in concrete, that is, rolled beams with bottom cover-plates only, and plate girders with heavy bottom and light top flanges, and much of the work has stood the test of severe service.

HENRY GOLDMARK,* M. AM. SOC. C. E.—The speaker is much interested in Mr. Hovey's remarks on the historical development of bridge design.† He has always felt that, in an applied science, the study of its evolution is not only of much theoretical interest, but also of direct value in understanding the principles underlying present practice. In the case of bridge engineering, the history of the different methods for taking account of the greater effect of the live loads, especially of train loads, in comparison with the dead weight, is of much interest. Their greater influence on the stresses was realized at an early stage of bridge development, and various methods of design, with this basic idea, have been used at different times.

The earlier methods were frankly empirical, whereas the later ones are based ostensibly on theoretical considerations. It can hardly be said, however, that even the latest practice has an exact scientific basis.

The bridge specifications of the past forty years reflect the varying views held at different periods. In the earliest specification issued by the late Theodore Cooper, M. Am. Soc. C. E., the cross-sections were determined by dividing the sum of the dead and live load forces in each member by a permissible unit stress. To allow for the great effect of live loads in short bridges, a lower stress was specified for such spans. Although many good bridges were designed under this specification, there was a serious objection to this method of proportioning, because of the fact that the unit stress changes suddenly at a given span length, say, from 8 000 to 10 000 lb. per sq. in. Therefore, for bridges differing only 1 ft. or 2 ft. in length, there might be a considerable difference in the permitted stress. This was clearly illogical and led to absurd results, as the longer bridge might actually have a smaller total weight.

* Cons. Engr., New York City.

† See p. 574.

The next step, based to some extent on theory, was to specify a unit stress for the dead load different from that for live loads. For instance, in Cooper's 1902 specification, the permitted live load stresses were half as great as the dead load stresses. This rule was evidently based on the theoretical consideration that a load applied instantaneously has twice the effect of a slowly applied load. It always takes an appreciable time before the stress in a given member, due to a train passing over a bridge, reaches its maximum value. Hence, the rule of halving the live load stress is not correct in theory, although undoubtedly safe in practice.

The next modification, and one that was widely used for some time, was to base the permitted stress on the maximum and minimum forces acting on the members at different times. This method was based on the formula of Professor W. Launhardt of the Royal Polytechnic School at Hanover, Germany. It was confessedly founded on the well-known series of experiments made by Professor Wöhler on the "fatigue" induced in iron bars under a large number of repeated loadings. Although the Wöhler tests were of great value, their application to bridge members, which act under entirely different conditions, was a doubtful step.

Even Professor Launhardt, with whom the speaker had occasion to discuss this matter while taking his course on bridge design, was quite ready to admit this. He was inclined to think that the more empirical methods of simply decreasing the permitted stress in long spans might be the better policy until further investigation should increase the knowledge of the subject. He somewhat whimsically called the exact determination of this question "music of the future".

In American practice, the Launhardt formula and the maximum minimum formula in its various forms have been generally discarded for many years. They have been superseded by the expression for "impact" introduced by the late C. C. Schneider, Past-President, Am. Soc. C. E., in his "Pencoyd" specification, and since then used in the specifications of the American Railway Engineering Association and other technical societies.

In these specifications, the effect of the live load is provided for in each member by an "impact" or "dynamic increment" which contains a term, L, equal to "the loaded length, producing the maximum strain in the member". This, of course, is proportional to the length of time during which the stress in the member gradually increases from zero to its maximum value.

The use of the term "impact" in the A. R. E. A. specification has been much criticized, especially as recent deformation measurements on bridges under service conditions, give somewhat discordant results and seem to show that the effect of live loads has been exaggerated.

It should be remembered, however, that in bridge engineering, as in most other fields of construction, great importance must be assigned to practical experience with structures in actual use, and that, at all times, theoretical considerations must give way to the test of service.

It is not believed that it would be wise to build lighter bridges than those now in use, especially in view of the doubt as to the weight of future engine

and train loads, until much more exact information has been obtained on such important questions as the strength of the newer alloy steels slowly coming into use, the strength of compression members, and the effect of live loads in bridge structures.

R. S. CHEW,* M. AM. SOC. C. E. (by letter).—In a discussion of a paper by Mr. James E. Howard,† the writer called attention to the fact that there is a definite eccentricity in all columns due to imperfections in the member. Although it is probably true that this fact is realized, it has not been given sufficient prominence in various column theories or formulas. The writer does not believe that any theoretical treatment of value can be developed, except in so far as to indicate the various factors that must enter as empiric constants in the working formula.

In order to indicate the action in a column, one is compelled to consider an imperfect member the material of which is not homogeneous in either a horizontal or a vertical plane. Let Fig. 12 indicate a column the imperfections of which are such that the center of resistance is removed from the center of gravity of the section by a distance, e, at the plane of the application of the load and that this distance, e, varies throughout the length of the column.

Let the line, R, indicate this resistance axis. Any number of curves could be drawn to indicate a possible resistance axis, but the one shown indicates the worst condition.

From this line, it may be noted:

(*a*) That all columns are crooked as regards resistance to axial compression.

(*b*) That the resistance axis does not lie in the plane of the principal axes, 1-1 or 2-2, except for circular sections.

Although all theoretical treatments are based primarily on the bending of the column, which, as may be seen, is caused by a crooked axis, yet no one has recognized the condition, (*b*), as the bending has been assumed to take place about Axis 1-1 or Axis 2-2.

Let a theoretical formula be written for axial compression in which:

l = Length (unsupported);
P = Axial load;
A = Area of section;
S = Maximum unit stress on extreme fiber;
r_1 = Radius of gyration about Axis 1-1;
r_2 = " " " " " 2-2;
n_1 = Factor for end conditions about Axis 1-1;
n_2 = " " " " " " 2-2;
$\frac{\phi r}{t}$ = A factor proposed by Lilly.

* Cons. Engr., San Francisco, Calif.

† "Some Tests of Large Steel Columns", *Transactions*, Am. Soc. C. E., Vol. LXXIII (September, 1911), p. 469.

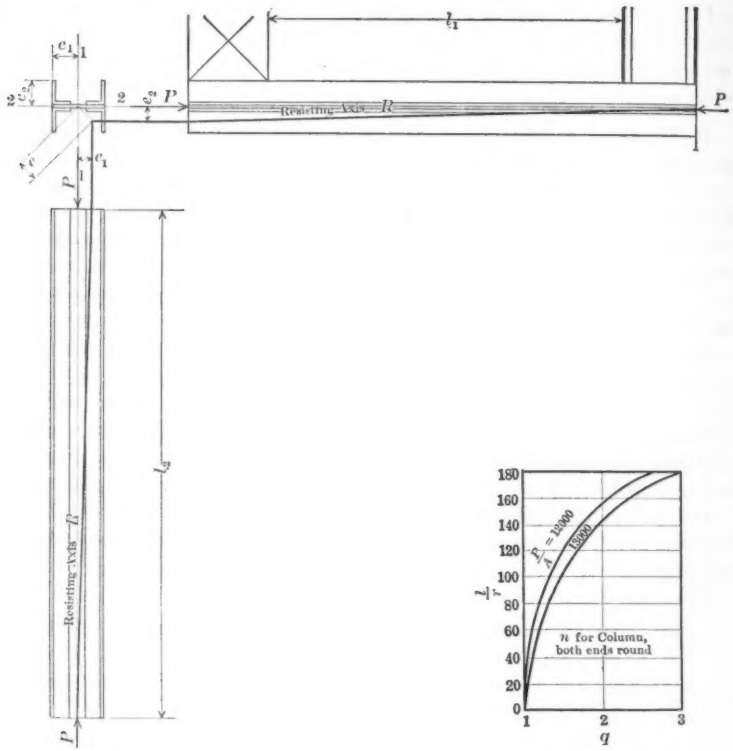


FIG. 12.

FIG. 13.

The formula would appear as follows:

$$S = \frac{P}{A}\left[1 + \frac{\phi r}{t} + \sqrt{\frac{e^2_1 c^2_1}{r^4_1}\sec^2\left(\frac{n_1 l_1}{r_1}\sqrt{\frac{P}{A E}}\right) + \frac{e^2_2 c^2_2}{r^4_2}\sec^2\left(\frac{n_2 l_2}{r_2}\sqrt{\frac{P}{A E}}\right)}\right]. \quad \text{(XVIII)}$$

Equation (XVIII) is of no value except as a study from which, by substitution of empiric constants, a working formula may be derived.

Some of the conditions under which a column acts, as regards Factor n, are as follows:

(1) Pin-ended on Axis 1-1; semi-fixed on Axis 2-2; $l_1 < l_2$; no positive moment, that is, the load is assumed as axial.
(2) Same as Condition (1), except $l_1 = l_2$; no positive moment.
(3) Semi-fixed on both axes; $l_1 = l_2$; " " "
(4) " " " " " ; $l_1 < l_2$ " " "
(5) " " " " " ; $l_1 = l_2$; positive moment, M.
(6) " " " " " ; $l_1 < l_2$ " " "

If r_1 is greater than r_2 and, assuming that the resistance axis does not lie in a plane, Axis 1-1 or Axis 2-2, then the resulting deformation is not that of bending alone, but is bending combined with twisting. Therefore, the combination of end conditions and lengths, l_1 and l_2, and the cross-section of the column must have an important bearing on the resulting stresses. Do either of the submitted formulas take any account of these conditions?

Working Formula.—If, by a series of tests, empiric values could be given to the factors, ϕ, e, and e_2, such that they would represent the average condition in commercial members carrying an axial load and having the various imperfections, Equation (XVIII) would be valuable.

In regard to the factor, n, let:

$$\sec^2\left(\frac{n\,l}{r}\sqrt{\frac{P}{A\,E}}\right) = q \ldots\ldots\ldots\ldots\ldots\ldots (XIX)$$

A series of curves may be drawn for assumed values of n and $\frac{P}{A}$ and for varying values, $\frac{l}{r}$, as shown in Fig. 13.

The formula will then be:

$$S = \frac{P}{A}\left(1 + \frac{\phi\,r}{t} + \sqrt{\frac{e^2_1\,c^2_1}{r^4_1}\,q_1 + \frac{e^2_2\,c^2_2\,q_2}{r^4_2}}\right) \ldots\ldots\ldots\ldots (XX)$$

which, with curves and given values of ϕ, e, and e_2, furnishes a working formula. Equation (XX) can be used in combination with the straight-line formula, that is, by determining a trial section by the following equation:

$$\frac{P}{A} = 15\,000 - 50\,\frac{l}{r}$$

and checking for S in Equation (XX).

The advantages of this formula are that it enables the designer to take care of positive moments and details, to check any damaged column, and to use his judgment between local and expert work for e.

H. M. MacKay,* M. Am. Soc. C. E. (by letter).—The one great object of a column formula is to enable the designer to predict the strength of a column; and possibly the diversity of opinion usually expressed arises from the consciousness that no formula yet proposed is even approximately satisfactory as a means of predicting the strength of a given built-up steel column of ordinary length. Euler's formula is quite satisfactory within the range to which it is applicable, as one may demonstrate with no more elaborate apparatus than a straight lath and a few weights. Rankine, as a reference to his works will show, had in mind solid or relatively thick-walled hollow struts of circular or rectangular section. Apparently, the only modern form which he visualized was a rectangular box-section made up of plates and angles. Both these eminent elasticians, quite justifiably under the circumstances, made the slenderness ratio, $\frac{l}{r}$, the only variable in their well-known formulas; and

* Prof. of Civ. Eng., McGill Univ., Montreal, Que., Canada.

it is perhaps due to the weight of their authority that engineers, in so far as their ideas are expressed in specifications, have continued to regard the slenderness ratio as the only variable. One does not forget the many suggested formulas in which other factors are included, but none of these, as far as the writer is aware, has come into current use, or has, in a broad way, passed the test of satisfactorily predicting column strength.

The most comprehensive series of tests on steel columns of reasonable size is that of the Society's Special Committee on Steel Columns and Struts, reported in 1918.* Three variables were included in the specimens tested: Slenderness ratio $\left(\frac{l}{r}\right)$, form or make-up, and thickness of metal. Considering only those tests in the case of which the complete schedule was carried out, eight different forms were tested, each form with three different values of $\frac{l}{r}$, and two different thicknesses of metal. The heavier sections, on the average, had roughly about twice the sectional area of the lighter sections. Three individual columns were tested for each slenderness ratio, form, and weight, making 144 tests in the group considered. Thus, 48 members of each slenderness ratio were tested, 18 of each form, and 72 each of the lighter and heavier sections. The results of these tests may be tabulated as follows:

A.—In respect of slenderness ratio:

$\frac{l}{r}$	Ultimate strength, in pounds per square inch.
50 (average of 48 tests).....	33 150
85 (" " " ").....	30 650
120 (" " " ").....	28 100

B.—In respect of form or make-up:

Form.	
5– 5*A* (average of 18 tests).....	33 700
8– 8*A* (" " " ").....	31 600
2– 2*A* (" " " ").....	31 000
10–10*A* (" " " ").....	30 400
4– 4*A* (" " " ").....	30 400
3– 3*A* (" " " ").....	30 200
1– 1*A* (" " " ").....	29 100
6– 6*A* (" " " ").....	28 600

C.—In respect of thickness of metal:

Light (average of 72 tests).....	32 200
Heavy (" " " ").....	29 000

The most cursory comparison of these results shows that, as far as these tests are concerned, the influence of thickness of metal on ultimate strength is as great as that of a variation in $\frac{l}{r}$ from 50 to about 95, a range wide enough to include practically all the seriously loaded columns in a steel railway bridge; whereas, the influence of form on the ultimate strength, even neglecting Form 6–6*A* as unusual and undesirable, is nearly 45% greater than that of

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1583.

the stated variation in slenderness ratio. Had latticed columns been included in this series of tests, the influence of form might well have been even more pronounced. Tests made for the Board of Engineers of the Quebec Bridge, in part by Mr. James E. Howard, and in part by the writer, indicated that members with solid transverse webs were 11 or 12% stronger, other things being equal, than those in which the ribs were connected by latticing only.

Although the Committee's tests bring out the influence of the thickness of metal in a way that no other tests do, any comprehensive group or groups of tests will indicate a large variation of strength due to form and detail, as compared with that due to a reasonable range in the value of $\frac{l}{r}$. When the strengths of a large number of columns are plotted against the slenderness ratio, the result is a broad belt of points through which a line can be drawn to suit any reasonable taste, or almost any formula. It is sometimes supposed that there is something erratic in the nature of column tests. For such a supposition, however, there seems to be little foundation. In the Committee's tests, the average extreme variation in each group of three members from the mean of the three, was 2.6 per cent. In the case of the Quebec Bridge tests referred to, the members are believed to have been among the largest, and certainly among the most complex, ever tested, the main section, in some cases, consisting of as many as thirty component parts. In this case, if in any, erratic results might be anticipated. Yet, in fifteen pairs tested, the average variation of a member in each pair from the mean of the two was only 1.54 per cent. In fact, it is the writer's experience, derived from several scores of tests, that compression members are rather more consistent in their behavior than large riveted tension members.

It is the writer's contention, therefore, supported as he believes by the whole body of available data, that the slenderness ratio, which in all formulas in general use is made the sole criterion for predicting the strength of columns of a given grade of material, is far from being the most important factor in the case of the great majority of heavily loaded columns. Approaching the question for the first time, and with an open mind, one would surely regard with astonishment the somewhat hair-splitting contentions in which individuals, and even committees, sometimes indulge, as to the precise function of $\frac{l}{r}$ to be used, whereas other and more important factors are ignored, except in respect to certain limiting clauses. Had equally unscientific methods been used in the calculation of tides, for instance, tide tables would have been based on the apparent diurnal motion of the moon. The consequent confusion of mariners, in Eastern waters, would hardly be greater than that of the designer who tried to correlate the strengths of various types of columns on the basis of their slenderness ratio.

It might be possible, as has been suggested, to devise different formulas for different types of column; but without much further investigation, agreement in this direction can hardly be expected. The form which it may be

thought investigation should take, will depend largely on the view entertained as to column action. The writer's experience in testing and studying the stress distribution in built-up structural members leads him to the view that, for the purpose of investigation, a column can be advantageously regarded as an aggregation of strips, each tending to yield in its own way, but restrained by mutual interaction. In solid and in thick-walled hollow sections, such restraint is practically complete, and the column acts as a unit. In such cases, if uniform material of a known quality and free from internal stress can be assumed, it should be possible to express the strength satisfactorily as a factor of $\frac{l}{r}$. In the more usual types, there will be a variation according to form and detail. If this view is correct, two corollaries seem to follow:

1.—That no method of analysis is likely to be satisfactory which assumes a linear variation of strain across the section, whatever relation between stress and strain may be assumed beyond the elastic limit, because such linear variation does not in general exist at the critical sections. Strain measurements show this to be true.

2.—That useful information regarding the laws of column action can be better obtained by an intensive study of a comparatively limited number of specimens, than by tests to destruction only, of a larger number. The latter seldom throw much light on the factors contributory to failure.

The possibilities of such intensive study are given on Fig. 14, on which is shown the stress distribution at certain sections of a column, consisting of four 2½ by 2½ by ¼-in. angles, 7 ft. 2 in. long, connected by tie-plates. The end-plates are ½ in. thick and project ½ in. beyond the angles, so as to take the load which is transmitted to the angles through the connecting rivets. The intermediate tie-plates are ¼ in. thick. The stresses at the various sections, as determined by mirror extensometers over 2-in. gauge lengths, are plotted on the cross-section as a base plane, and the resulting solid is then shown in isometric projection. The distribution of stress can thus be seen at a glance. No extensometer readings were taken on contiguous surfaces of angles or plates, so that, in such cases, the stress distribution is, to some extent, a matter of conjecture. However, in all cases, the stresses as plotted check the applied loads within 4 or 5 per cent. Section 1 is just within the first rivets nearest the ends of the angles. Most of the load remains in the plates, the angles taking only the amount transmitted by one rivet in each. At Section 2, the angles take an additional amount of load transmitted by a second rivet. The stress distribution remains far from uniform, certain fibers being in tension. At Section 5, where the angles take all the load, the stress distribution is fairly uniform, but this uniformity is disturbed again, where, as at Section 4, a tie-plate is encountered. Notwithstanding the greater sectional area at Section 4, the maximum fiber stress in the angles is about 45% greater there than at Section 5. The stress distribution at all sections remained, to a remarkable extent, the same at all loads from 20 000 to 80 000 lb.

The stress survey, of which a few of the results are here shown, was made under the writer's direction by the late Lieut. H. G. S. Delepine. Such

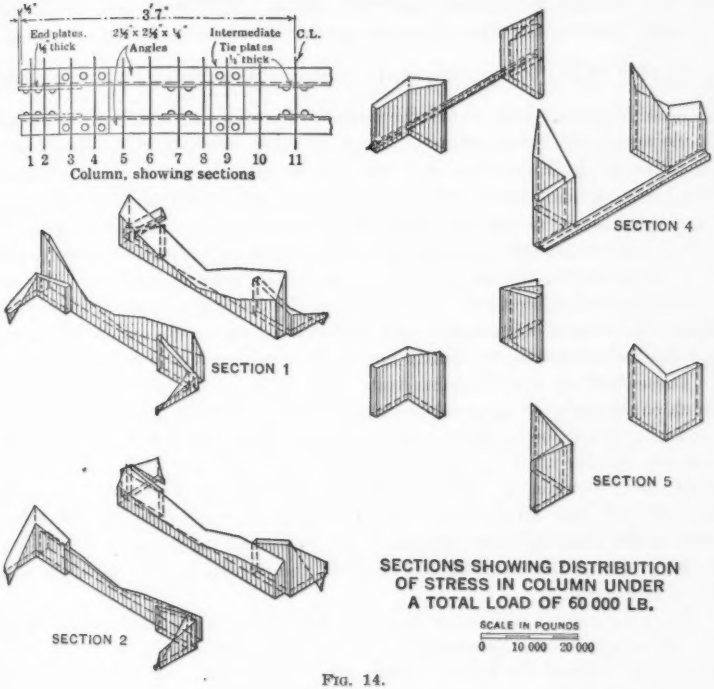


FIG. 14.

surveys are laborious; but the writer believes that only by the study of typical columns in some such manner can the Engineering Profession successfully take up the challenge which the present unsatisfactory knowledge of column action places squarely before it.

ALFRED S. NILES, JR.,* JUN. AM. SOC. C. E. (by letter).—The writer has been much interested in the discussion of the column formula. In designing aeroplane structures, it is necessary to use very slender columns and low factors of safety. On account of these two characteristics of aeroplane structural members, much research work has been done on columns of both wood and steel, the results of which are not well known to the Profession. Although the columns used in aeroplanes are small, they cover a wide range of slenderness ratios, and give interesting information on the subject of columns in general.

The most important result obtained is that the Euler formula will give the failing load for a truly centrally loaded column, provided the Euler load is less than the failing stress of the material in direct compression. Some of the most interesting tests have been made on spruce columns, in which, owing to the lack of homogeneity of wood, one might expect inconsistent results. The average crushing strength of aeroplane spruce is about 6 000 lb. per sq. in.,

* Aeronautical Structural Engr., Research Dept., McCook Field, Dayton, Ohio.

and the average modulus of elasticity about 1 800 000 lb. per sq. in. The ultimate stress by the Euler formula equals the crushing strength at values of $\frac{l}{r}$ between 50 and 60 for pin-ended columns. A series of tests was recently made at McCook Field, under the writer's direction, in which all struts with slenderness ratios of 60 or more failed at values slightly below the ultimate, according to the Euler formula. Struts with slenderness ratios of 50 and 40 failed at practically the same load as short pieces cut from the ends of the same specimens. Great care was taken to prevent eccentricities of loading, and if any measurable deflection was obtained under about two-thirds of the expected ultimate, the column was reset in the testing machine. This procedure eliminated nearly all eccentricities, whether due to lack of homogeneity of material or any other cause. The difference between the actual failing load and the Euler load was evidently due to the small eccentricity remaining.

These McCook Field column tests were made under laboratory conditions which could never be approached in engineering practice. Their value lies in proving that the lower limit of the range in which the Euler formula applies is much smaller than had previously been believed, and that the reason for using other formulas for short struts was not because the Euler formula did not hold for such struts, but because the effect of eccentricities becomes so great in the short columns that the Euler formula is no longer a safe approximation. The Euler formula does hold if the conditions are those for which it was developed.

The real problem in column design is that of making a reasonable allowance for the effect of the eccentricities which are, partly at least, unknown in magnitude. When the Euler formula gives unit stresses of less than one-half the yield point of the material, the effect of the eccentricities usually encountered is so small that it may safely be neglected as taken care of in the factor of safety. This puts the lower limit of the practical range of the Euler formula at about $\frac{l}{r} = 120$ for structural steel. As bridges have few, if any, structural members with such high slenderness ratios, the Euler formula is of little use to the bridge designer.

For columns with slenderness ratios in the range of bridge practice, allowance must be made for the effect of eccentricities, and it is most profitable to view the proposed column formulas in this light. As the eccentricity and degree of restraint of the ends of the actual column are impossible to determine accurately, it is necessary to apply empirical constants in any practical formula for columns in this range. All the formulas that are proposed for short columns have these empirical constants, even though they also have a more or less theoretical basis. This applies to the Rankine formula, and the many "eccentricity" and "secant" formulas proposed.

The column formula which should be used for practical design is the one which combines, in the greatest degree, ease of application, and agreement with test results. These advantages belong to the parabolic and straight-line formulas. The studies of A. Ostenfeld, in 1898, on the better known experimental

work, by the method of least squares, indicated that these formulas give the smaller and the Rankine and eccentricity formulas the greater mean errors.* As these formulas are not only the most easily applied, but also represent the test results the most accurately, the writer can see no reason for the attempt to use complex formulas like those proposed by Mr. Chew† and Mr. Hunley.‡ These formulas have a more theoretical basis than the parabolic formulas and appear to be more accurate, because they allow for the initial eccentricity. This appearance of accuracy, however, is illusory, as the initial eccentricity must be assumed or found empirically, and as Ostenfeld has shown, the results are not in as close agreement with the tests as those of the simpler and frankly empirical formulas.

In aeroplane design the parabolic formula is used exclusively for short columns, with excellent results, for both wood and steel members. It is easy to apply, and the constants have a definite connection with the properties of the material.

HENRY B. SEAMAN,§ M. AM. SOC. C. E. (by letter).—Several of the larger matters of general importance, having great influence in the design, such as engine loading, impact, and column formulas, have received prolonged consideration by the Special Committee on Specifications for Bridge Design and Construction, and it seems proper that these subjects should be presented more fully than is shown in the Tentative Specifications.

Engine Loading.—The subject of engine loading was considered from various standpoints. It was recognized that engine concentrations had increased far beyond the loads for which this distribution was intended originally, and that although this typical engine had simplified the calculations for design, a still further simplification was desirable, if practicable. To this end was suggested a uniform load with single concentration, and other methods, but when all things were considered, particularly the established tables and diagrams now in use, it was evident that there were no advantages over the present general practice. Furthermore, a study of the chart of engine moments and shears, as already considered by the American Railway Engineering Association,‖ showed that the typical engine distribution, even with the increased loading, gave satisfactory results as compared with the group of engines in actual service on several railroads. Whether it may yet be possible to devise a new system of loading which will conform even more closely to the engines in actual use, may be a subject for broad discussion. However, with this heavy loading, and the further provision in the Tentative Specifications for overload, it may be questioned whether it is still necessary to add the double concentration of passenger engines. This is particularly true if proper provision is made beyond 100%, for impact on spans of less than 30 ft.

Impact Formula.—The first impact formula which came to the writer's attention was presented by the late C. C. Schneider, Past-President,

* Salmon, "Columns", p. 231.
† See p. 583.
‡ See p. 568.
§ Cons. Engr., New York City.
‖ *Proceedings*, Am. Ry. Eng. Assoc., Vol. 21, p. 571.

Am. Soc. C. E., in his specification of 1887, in which $I = 0.7 + \frac{5}{L}$. This formula was introduced to simplify the design of bridges, as compared with the application of the Launhardt formula which at that time was coming into general use. Eight years later (1895), the Schneider formula was replaced by that in which $I = \frac{300}{L + 300}$, sometimes known as the "Pencoyd formula," which was introduced into the bridge specifications of the American Railway Engineering and Maintenance of Way Association of 1905, 1906, and 1910.

The subject of impact has received more consideration and prolonged discussion by the Committee than any other item of the specification. The results of the tests* made by J. E. Greiner, M. Am. Soc. C. E., on the Baltimore and Ohio Railroad Bridges, of those† made by F. E. Turneaure, M. Am. Soc. C. E., and of those‡ made more recently in England by Maj. A. Mount, R. E., were before the Committee.

The tests of the American Railway Engineering and Maintenance of Way Association, unfortunately, did not include spans less than 35 ft. in length, but the results given by Maj. Mount, shown in Fig. 15, largely supply the deficiency, although the English engine, with its inside connections, may not be directly comparable with the American engine which has its connections outside the driving wheels and, possibly, gives greater impact from unbalanced forces. In spite of this favorable consideration of the English engine, it is noticed that in Maj. Mount's results a rail bearer, composed of two halves fastened together, of 7 ft. 6 in. loaded length, gave an impact of 159% on one half and 90% on the other half, making an average of 125%; also, a loaded length of 9 ft. 8 in. gave an extreme impact of 143%, with an average of 92 per cent. Such results are rather startling to those who have made no provision beyond 100% in very short spans, but they are what should have been expected from the hammer blow of unbalanced drivers on short, rigid beams. It approaches the hypothetical condition of the "irresistible force" meeting the "immovable body". Such short spans are rarely used, and involve little extra expense, but proper provision should be made for the condition. The tendency toward a sharp increase of impact in very short spans is recognized to some extent by every impact formula that has been proposed, except the one recently adopted by the American Railway Engineering Association, in its 1920 specifications, namely, $I = \frac{300}{300 + \frac{L^2}{100}}$, which gives practically the same impact for all spans of less than 25 ft. The Committee recognized the advisability of providing for greater increase of impact on short spans—125% was incidentally mentioned for 0 span, and 100% for spans of 30 ft.

The tests show that impact for long spans—700 ft., or more—may be considered to be practically negligible. Furthermore, it is desirable to adopt a

* *Proceedings*, Am. Ry. Eng. and M. of W. Assoc., Vol. 6 (1905).
† *Proceedings*, Am. Ry. Eng. and M. of W. Assoc., Vol. 12, Pt. 3 (1911).
‡ *Engineering News-Record*, October 20th, 1921.

formula which will be applicable to spans of all lengths. There is no reason why the change from minimum impact to maximum impact should not be gradual and continuous; conditions do not reverse for short spans; but it must be remembered that the whole subject is empirical, and any formula adopted now may be modified by future experiments.

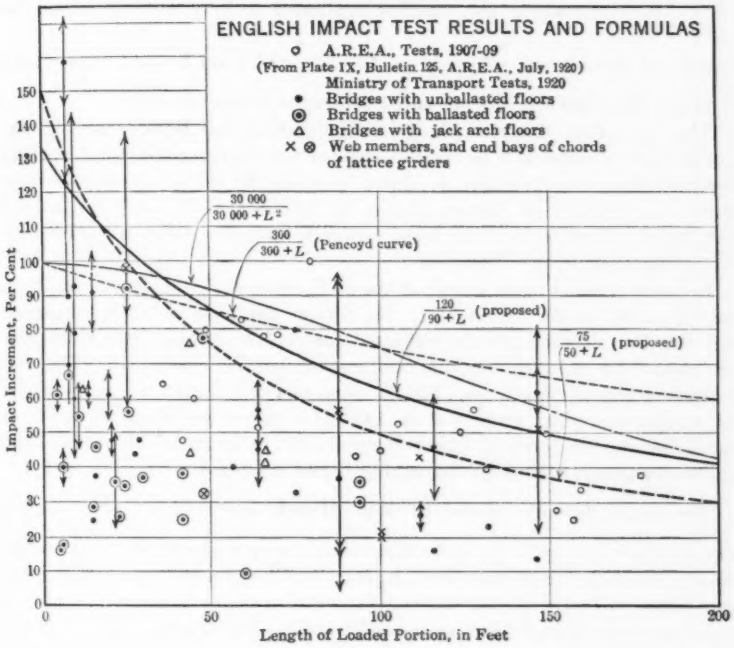


FIG. 15.

About 1905 it was the custom to use special specifications for each bridge designed, conforming to the particular conditions of span and loading. There were three or four different specifications for short spans, and another entirely different specification for long spans. There seemed to be no sharp line of demarcation between short and long spans, although it was generally assumed to be about 300 ft., at which the dead load approached or exceeded the live load. A single standard specification which would be applicable to spans of all lengths seemed to be eminently desirable. The fundamental difference in the design of short and long-span bridges was the relative action of the live load, and this should be stated in a "formula for equivalent static stress", as it was then expressed. With the use of such a formula all live load stresses might be reduced to equivalent static stresses and the allowable unit stresses modified accordingly.

At that time the impact tests of the American Railway Engineering and Maintenance of Way Association had not been made, and the only guide for

such a formula were the tests made by Mr. Greiner, previously mentioned, and the weights of bridges which had been built. The fact that impact on long spans would be negligible was already evident from the long spans erected in New York City, and has since been indicated by the tests; while, for very short spans, impact of 100%, or more, seemed to be advisable. The formula, $I = \frac{300}{L + 300}$, already in use, was not applicable to very long spans, and the formula of the quarter ellipse, $I = 125 - \frac{1}{8}\sqrt{2\,000\,L - L^2}$, was finally adopted. This formula was presented before the Society in 1912.*

The tests of the American Railway Engineering and Maintenance of Way Association, as published in its *Proceedings* for 1911, constituted a striking confirmation of this formula, as shown by Plate X† of the discussion of the writer's paper by S. W. Bowen, Assoc. M. Am. Soc. C. E. A more detailed examination of several of the tests, however, indicates further confirmation than was noticed in the discussion.

In Table 6-o the north girder of the 80-ft. span shows an impact of 122%, and the south girder shows 74%; the former again seems to be abnormal, while the latter comes near the elliptical curve. These three tests which were not plotted in the original diagram, are shown by circles (o) (Fig. 16). This curve runs from 0% for 1 000-ft. spans to 125% for 0 span, and provides for impact somewhat greater than 100% for spans less than 20 ft. It is the oldest impact formula which is applicable to spans of all lengths, and deserves consideration unless something better is offered.

The impact formula offered by Maj. Mount, namely, $I = \frac{120}{90 + L}$, gives very satisfactory results for short spans, and for spans less than 30 ft., makes proper provision of more than 100% impact; but it seems to be too high for very long spans.

In regard to impact, little more need be said. The diagram, Fig. 17, shows the striking agreement between the formula,

$$I = 125 - \frac{1}{8}\sqrt{2\,000\,L - L^2},$$

and the tests.

It has been suggested that the new A. R. E. A. impact formula is intended only for spans of 300 ft., or less. This limitation would exclude it from this specification, which covers spans of all lengths. By inspection of Fig. 16, it will be noticed that the new A. R. E. A. formula is too high even for short spans of 40 to 120 ft., where plate girders, which need no excess of strength, are used, whereas, for 120 to 300 ft., where truss spans, which need the fullest provision for loading, in anticipation of the allowable overload stress of 24 000 lb. or more, are used, the formula is deficient, notably so for spans of 210 and 300 ft. If the test, giving 22% impact for the lower chord of the 440-ft. span at Bellefontaine,‡ is plotted, it is found again to be deficient.

* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 340.
† *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 355.
‡ As shown in Column 3, Location 1822, Table 6-*bm*, of the A. R. E. A. Report of 1911.

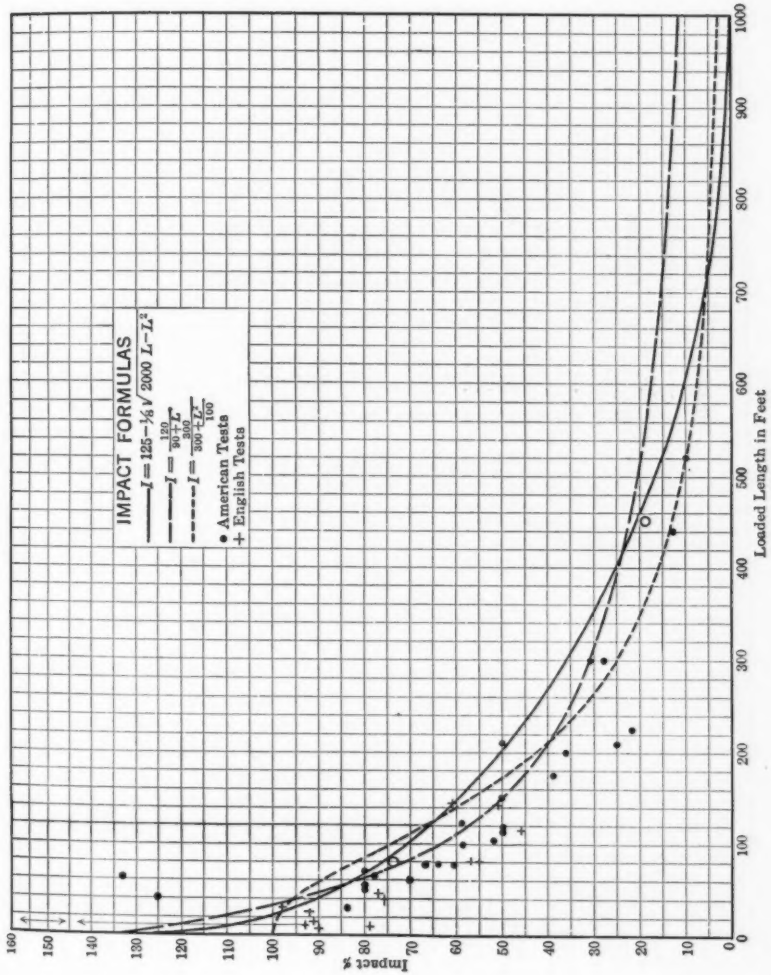

IMPACT FORMULAS
$I = 125 - \frac{1}{8}\sqrt{2000\,L - L^2}$
$I = \frac{120}{90 + L}$
$I = \frac{300 + L^2}{300 + L^2}$
• American Tests
+ English Tests
Impact %
Loaded Length in Feet
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
0
100
200
300
400
500
600
700
800
900
1000

FIG. 16.

L.	I%	L.	I%	L.	I%	L.	I%
1	119.41	25	97.32	150	59.15	390	25.95
2	117.10	26	96.68	160	57.18	400	25.00
3	115.32	27	96.15	170	55.28	410	24.07
4	113.83	28	95.63	180	53.45	420	23.17
5	112.52	29	95.12	190	51.70	430	22.29
6	111.33	30	94.61	200	50.00	440	21.44
7	110.23	35	92.22	210	48.35	450	20.60
8	109.22	40	90.00	220	46.78	460	19.79
9	108.27	45	87.91	230	45.24	470	19.00
10	107.37	50	85.97	240	43.76	480	18.23
11	106.51	55	84.12	250	42.32	490	17.48
12	105.69	60	82.35	260	40.92	500	16.75
13	104.91	65	80.67	270	39.57	510	16.03
14	104.16	70	79.06	280	38.25	520	15.34
15	103.43	75	77.51	290	36.97	540	14.01
16	102.73	80	76.01	300	35.73	560	12.75
17	102.05	85	74.57	310	34.52	580	11.56
18	101.39	90	73.17	320	33.35	600	10.41
19	100.75	95	71.82	330	32.20	650	7.91
20	100.13	100	70.51	340	31.09	700	5.76
21	99.52	110	68.00	350	30.01	750	3.97
22	98.92	120	65.63	360	28.95	800	2.53
23	98.34	130	63.37	370	27.93	850	1.41
24	97.78	140	61.21	380	26.92	900	0.63

FIG. 17.

The selection of the proper impact formula, however, is only part of the problem of impact on bridges. The relative effect of dead load may be important, but as yet there are no tests for comparison. The effect from multiple tracks must also be considered. The provision that the load for multiple tracks shall be reduced 10%, 20%, or 30%, for two, three, or four tracks, respectively, is crude. The conditions should be susceptible of better analysis.

It is known that the impact of the tests is due mainly to the blow of the unbalanced drivers and to the synchronism of the interval of the hammer blow with that of the vibration of the span. The same synchronism from a second train is manifestly impossible. The second train would be more likely to destroy the synchronism of the first train than to augment it. Under these circumstances there should be but little additional impact from the second train. In estimating impact from the second train, it would be better, after using full impact from the first train, to use the aggregate loaded length of both tracks for the value of L, somewhat as was done in the A. R. E. A. specification of 1910. For tracks in reverse operation (as the second track in two-track bridges, and the third and fourth tracks in four-track bridges), only the uniform train load, without impact, should be used; and, in order to avoid unnecessary refinement in calculations, only full panel loads at the panel points should be considered. All bridges should be designed for possible "left-hand" train service.

In selecting an impact formula, it should be remembered that a stress of 24 000 lb. per sq. in. is provided in cases of overload, and any formula which may be adopted, therefore, should cover extreme conditions. To be able to maintain a bridge intelligently is quite as important as to construct one; in fact, engineers should always bear in mind the contingency of maintenance in bridge design.

Column Formula.—In the early days of iron bridges, the practice was to proportion columns by one of three formulas, depending on the assumed condition of the column ends, whether fixed at both ends, fixed at one end and round at the other, or round at both ends. Later, it was considered advisable to use only one formula, that for both ends round, as the end conditions of columns in a structure are uncertain, and the column may be subjected to many unforeseen conditions. For columns with round ends the formula then used was of the Rankine-Gordon type, which was considered to be so approximate, and yet so safe, that a straight-line formula might give equally satisfactory results within the limits of column length usually found in bridge design. It was a new thought, a change from the monotonous routine of years, and a recognition that engineers need not be confined to exact proportioning where unknown conditions were so prevalent. Tabulated values of r for various sections simplified its application, and the greater conveniences of the older formula—of the Rankine-Gordon type—were soon forgotten. The straight-line formula, introduced into the specifications of the American Railway Engineering and Maintenance of Way Association, of 1905, 1906, and 1910, was $p = 16\,000$ lb. $- 70\,\frac{l}{r}$.

A great number of tests of full-sized columns have been made since 1880, and these tests showed the surprising result that pin-end columns do not act as round ends, but rather as flat ends—that the bearing on the pin is apparently sufficient to fix the direction as though the column ends were flat. The tests on steel tubes also indicated that the square-end columns and the fixed-end columns gave practically the same results.

The most systematic and complete tests that have been made, except for extreme lengths, are those of the United States Bureau of Standards, for the Society's Special Committee on Steel Columns and Struts. Flat ends were used in all cases, and, in order to produce equivalent round-end tests, it is necessary to plot the results to half length.

The preference for the straight-line formula was based on its simple form, but the Special Committee on Steel Columns and Struts went one step further and almost abolished the column formula altogether. It proposed a uniform allowable stress of 12 000 lb. per sq. in. for all columns, irrespective of length, up to $\frac{l}{r} = 80$, and beyond that length the allowable stress was reduced by $100 \frac{l}{r}$, and no column was used longer than $\frac{l}{r} = 120$. Could anything be simpler? This simple formula was never actually adopted, but its influence is shown in the new straight-line formula of the American Railway Engineering Association specifications of 1920, which allows a uniform stress of 12 500 lb. per sq. in. on all columns, irrespective of length, up to $\frac{l}{r} = 50$, and beyond that length reduces the allowable stress by $50 \frac{l}{r}$. This formula is as simple as that proposed by the Special Committee on Steel Columns and Struts, except that only one-half the value of $\frac{l}{r}$, as proposed by the Committee, is deducted for long columns. Both these formulas neglect the well established rule that the maximum allowable column stress should be placed at $\frac{l}{r} = 40$, which corresponds to $\frac{l}{d} = 12$ for solid rectangular sections. The upper limit of 12 500 lb., as compared with the basis of 16 000 lb. for tension, however, seems to be a useless waste of material, and its application may involve needless expense to railroads. It will condemn many existing bridges which have been well designed under the previously accepted column formulas. It is unscientific, and is particularly objectionable in long spans, where all useless dead weight should be eliminated. The confidence in a straight-line formula appears to be wavering. The Canadian Engineering Standards Association has abandoned it altogether and adopted the "parabolic formula", $I = 12\,500 - \frac{1}{4} \frac{l^2}{r^2}$, which, although it is as purely empirical as the straight-line formula, may be used in maintenance for bridge rating. Is it not an opportune time for engineers to return to a reconsideration of the entire

subject? Each individual may formulate his own practice, but a specification should be based on scientific principles.

The Euler formula for stress due to flexure in long columns has remained unquestioned for 160 years, the only criticism being that it does not provide for the direct stress to which all columns are subjected. Engineers cannot do better than accept this as a basis for a formula for combined stresses.

The substance of the following clause is found in every bridge specification:

"Members subject to both direct and bending stresses shall be proportioned so that the combined fiber stress shall not exceed the allowable stress specified."

Long columns, at the instant of rupture, are subjected to both direct and bending stresses. The direct stress is proportional to the applied load, P. The bending stress, according to the Euler formula, is inversely proportional to $\frac{l^2}{r^2}$; thus, according to Euler:

$$P = A\,\pi^2\,E\left(\frac{r}{l}\right)^2 = c\left(\frac{r}{l}\right)^2 = \frac{c}{\frac{l^2}{r^2}} = \frac{1}{\frac{l^2}{c\,r^2}}$$

The formula for the combined stresses may then be written:

$$Y = P + P\left(\frac{l^2}{a\,r^2}\right) = P\left(1 + \frac{l^2}{a\,r^2}\right) \qquad \text{(XXI)}$$

from which may be found the theoretical fiber stress in the column for any given load, P. This is of great importance in rating the strength of old bridges in service, and is invaluable to the maintaining engineer.

From Equation (XXI) the modified Euler column formula is derived, which includes provision for direct stress:

$$P = \frac{Y}{1 + \frac{l^2}{a\,r^2}} \qquad \text{(XXII)}$$

where P is the applied load, Y is the yield point of the material at the outer fiber, and a is a constant to be found by experiment.

In long columns the yield point of the material is the ultimate strength of the column, because, at the instant of yielding, the flexure increases rapidly, and the resulting increase of the bending moment in the column will cause complete failure if the testing machine follows with full load as rapidly as the column collapses.

This formula has been in use since the early days of iron bridge design. It is applicable to columns of all lengths, and may be used in maintenance as well as in construction.

The results of tests on full-sized columns made by the U. S. Bureau of Standards for the Special Committee on Steel Columns and Struts are plotted on Fig. 18, on which also is shown the comparative value of the old and the new straightline formulas of the American Railway Engineering Association and of the modified Euler formula with relation to those tests. The coefficient, a, of the last named formula has been placed at 13 500.

For convenience in plotting the allowable stresses on columns by Equation (XXII), Table 9 of the Final Report of the Special Committee on Specifications for Bridge Design and Construction on Steel Railway Bridge Superstructure,* may be used.

The design and construction of steel bridges is still in a transition stage. The loading has been increased until it would seem that the limit had been reached, but improvements in the material are still to be expected. Steel 50% stronger than the structural steel usually specified is already in use, and there is a possibility, even a probability, that a still stronger steel may soon be offered. When this comes the formula previously mentioned will remain unchanged, except that the numerator will be increased in proportion to the increased yield point of the new material.

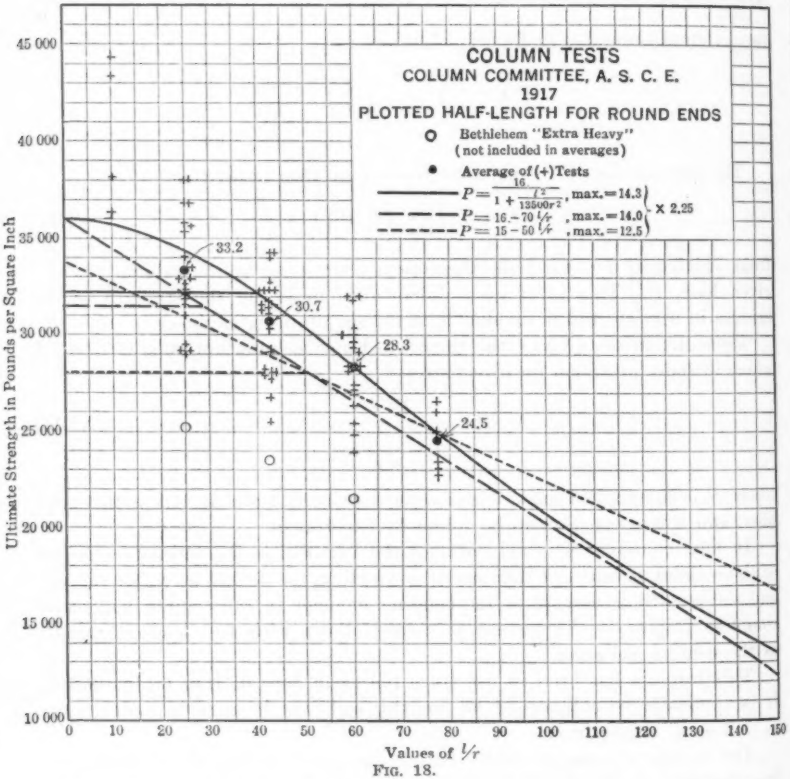


FIG. 18.

The accuracy and value of the modified Euler formula need no explanation. It is the Rankine-Gordon formula of long service. Of the two American Railway Engineering Association formulas, the old one, $P = 16\,000 - 70\,\frac{l}{r}$,

* See p. 499.

is parallel in direction to the averages of the tests, and with a slight increase of the factor of safety would practically conform to those tests; the new formula of 1920, $P = 12\,500 - 50\,\frac{l}{r}$, allows greater stresses for the longer

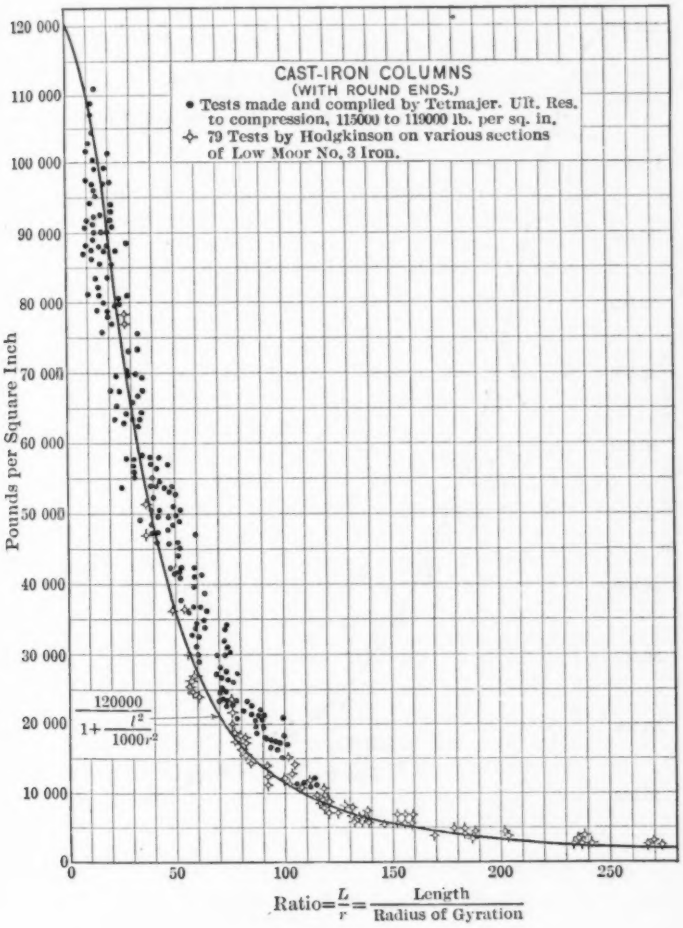


FIG. 19.

columns. Neither of these straight-line formulas can be used definitely to obtain the fiber stress to which a long column may be subjected in service. The only objection to the modified Euler formula seems to be that, for convenient use, it requires a table of values.

In Table 10 of the Final Report of the Special Committee on Specifications for Bridge Design and Construction on Steel Railway Bridge Superstructure,* is given values of the coefficient $\left(1 + \frac{l^2}{13\,500\,r^2}\right)$, Equation (XXI), for values of $\frac{l}{r}$ from 40 to 250.

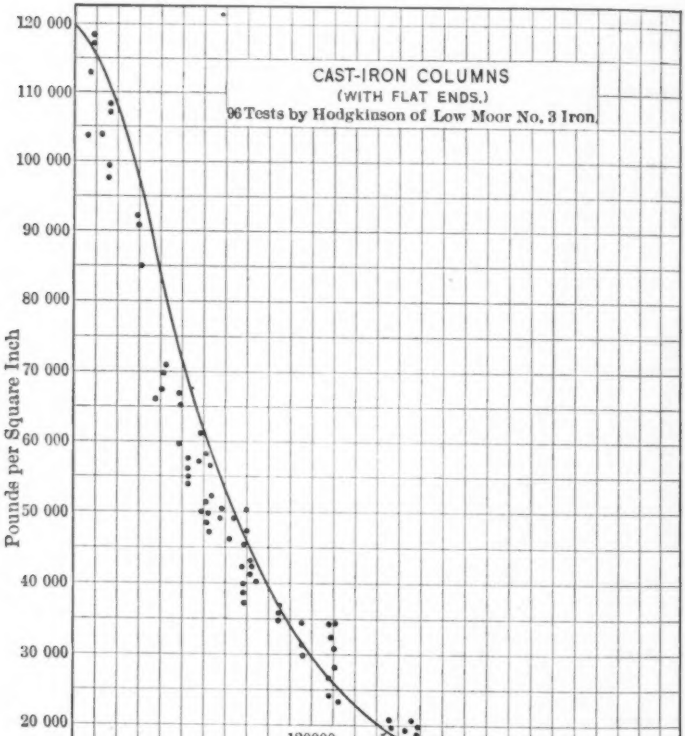


FIG. 20.

Since the early days of bridge design, the longitudinal force has been considered as due to a coefficient of friction of 20 per cent. A thorough investigation of this subject, some years ago, convinced the writer that this was an

* See p. 499.

error, and that the specification was honored more in its breach than in its observance. In his specification, previously mentioned, and for reasons then expressed, a coefficient of 10% was adopted, and this, he believes, is higher than necessary.

The strongest claim that has been offered for any practicable column formula, whether it is the straight-line or the parabola, is its agreement with the results of full sized tests. The Schwarz, the Gordon, the Rankine, or the Euler-modified, or by whatever name it may be known in its long line of endorsement, surpass any of the others in comparison with the tests, as shown in Fig. 18. The correctness of the general formula is strikingly depicted in the diagrams, Figs. 19 and 20, of tests by Tetmajer and Hodgkinson, on cast iron.* The homogeneity of the material in these tests, and the freedom from irregularities due to workmanship, make the comparison of the formula with the tests particularly instructive as illustrating the law of column flexure and failure. This formula is of a rational, if indeed not of a completely theoretical, derivation, and is the oldest and best established formula in general use. The following paragraphs seem worthy of consideration:

The fields of the American Railway Engineering Association and the American Society of Civil Engineers are distinct, as has been suggested by Mr. Guppy and others. The work of this Society is toward professional suggestion, rather than business authority. The authority, as well as the responsibility, must rest with the individual engineer.

The suggestion that a bridge should be proportioned for its ultimate load is really the crux of bridge design. It was with this in view that the writer offered a specification to the Society in 1912,† in which the loading of that time, E-40, was increased 25%, to E-50, and, at the same time, the basic unit stress of 16 000 lb. was increased 25%, to 20 000 lb., which was then considered to be the highest stress at which a bridge should be maintained. The recent adoption by the A. R. E. A. of 24 000 lb., or more, as the safe limit, has been accepted with some misgiving, but the fact that the maximum assumed loadings are seldom reached in service is probably the saving feature in the use of this high stress. Although one may prefer that a bridge be proportioned for the ultimate load, a direct specification to that effect does not seem to be practicable under the present system of railroad management. The clause for overload only partly covers this condition, and leaves waste material in the chord sections. It would be better to permit an increase of 50% in the allowable stresses specified, provided the structure, so designed, would carry a like percentage of overload.

Wind Pressure.—The structure shall be designed to resist a wind pressure, acting in either direction horizontally, of 30 lb. per sq. ft. of surfaces, as seen in elevation, the floor system, considered solid area, one truss complete, and one-half the surface of all remaining trusses (but never less than 200 lb. per lin. ft. of loaded chord, and 150 lb. per lin. ft. of unloaded chord), as well as that of a train 14 ft. high. The pressure on the train, as well as one-half the pressure on the structure, shall be considered as a moving load.

* Plotted by A. J. Malukoff, Assistant Engineer, Department of Bridges, New York City.
† *Transactions*, Am. Soc. C. E., Vol. LXXXV (1912), p. 340.

If a wind pressure of 50 lb. per sq. ft. of unloaded structure would produce greater stresses than the loading specified in Article 109, such stresses shall be used.

In calculating the stresses due to wind forces on viaduct towers, the viaduct shall be considered as loaded on the leeward track with empty cars, of weight only sufficient to prevent overturning, with a wind pressure of 500 lb. per lin. ft., acting 7 ft. above top of rail.

The requirement in some specifications that provision shall be made for a lateral force of 10% of the specified train load on one track seems very excessive. The centrifugal force of a train moving at a speed of 30 miles per hour around a 9° curve is less than 10% and no such condition could exist for a train moving on a straight track.

Unit Stresses.—In proportioning the several parts of the structure the allowable stresses shall be increased 50%, and the structure designed to carry a live load 50% greater than that specified.

Allowable Pressure on Soils.—Wherever practicable, special tests of the bearing value of the soil will be made, but in the absence of more definite information, the following pressures may be allowed:

	Kips per square foot.
Sound ledge rock	30
Hardpan, or compact gravel	16
Coarse sand, or gravel	12
Clean sand, or dry clay	8
Clay (moist)	4
Soft alluvial soil	1

On deep foundations allowance may be made for buoyancy and friction.

Allowable Pressure on Wooden Piles.—Piles shall not be placed closer than 2 ft. 6 in., centers, and will be generally designed to sustain 30 000 lb. per pile. Their supporting power, when driven, will be estimated as follows:

(*a*).—When the pile is driven to refusal and supports the load by end bearing only, without friction, allow 1 200 lb. per sq. in. of smallest cross-section, properly reduced for column length when not braced.

(*b*).—When driven to resistance by hammer blows,

$$p = \frac{2\,W\,h}{d + 1}$$

(*c*).—When driven by water jet, or in soft material,

$$p = (S\,a + R)\,L$$

in which:

p = safe load per pile, in pounds;
W = weight of hammer, in pounds;
h = height of fall of hammer at last blow, in feet;
d = penetration under last blow, in inches;
S = skin surface of pile in ground, in square feet; and
L = length of pile in ground, in feet.

Net Sections.—Net sections shall be used, in all cases, in calculating tension members, and, in deducting rivet holes they shall be taken as $\frac{1}{8}$ in.

larger than the nominal diameter of the rivet. In calculating net sections having rivets staggered, all rows shall be deducted unless arranged so that the net section along a zigzag line, taking all distances on the diagonal as 90% of their value, exceeds the corresponding net section across the plate.

It is interesting to learn, as stated by Mr. Goldmark,* that Launhardt did not take his own formula seriously and was surprised that it should have been adopted so extensively in the United States. The subject was thoroughly reviewed by the speaker, from the experiments to the final derivation of the formula, and the results presented to the Society in the form of a paper.† The conclusion reached was that the Launhardt formula was not a demonstration, and the tests themselves justified the inference that the effect of an indefinite number of applications of a load was about the same as that of the single application of a load at double this amount. This is the basis on which Cooper wrote his later specification, making the allowable live load stresses equal to one-half the allowable dead load stress. Similarly, it was seen that reversed stresses, indefinitely repeated, had the effect of the sum of these stresses if applied in one direction.

Methods of bridge design, since that time, have been entirely changed. It is recognized that a load, within the elastic limit, or yield point, may be applied an indefinite number of times without practical injury to the material from fatigue. Engineers, therefore, have sought to find the greatest stress effect of an applied train load, and, for this purpose, impact tests have been made. Whether or not those tests show the result of impact, vibration, etc., they show the greatest possible intensity from live load, and when the maximum intensity of live load is known, and the allowable stresses are kept well within the elastic limit, the assumptions provide for fatigue, and are safe.

When stresses up to 24 000 lb. per sq. in. are maintained, the limit of safety is nearly reached, considering that there are still some unknown secondary stresses. Therefore, the impact formula, so-called, should cover the most extreme conditions known. The tests which were shown, occur rarely, but they do occur, and it is just that contingency which should be met. This is the basis of present-day bridge design.

The fact that a bridge has stood the test of service does not necessarily mean that it has been economically designed or properly proportioned.

A bridge of heavy, wasteful design may show a good service record, yet another which is lighter in weight, but more rigid under traffic, may produce an equally good record in service.

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* See p. 582.

† "The Launhardt Formula and Railroad Bridge Specifications," *Transactions*, Am. Soc. C. E., Vol. XLI (1899), p. 140.

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Paper No. 1520

LOCOMOTIVE LOADINGS FOR RAILWAY BRIDGES*

By D. B. Steinman,† M. Am. Soc. C. E.

With Discussion by Messers. C. D. Purdon, Robert C. Strachan, Victor H. Cochrane, Albert Lucius, B. A. Worthington, G. H. Gilbert, J. A. L. Waddell, Gustav Lindenthal, R. A. Caughey, J. C. Ralston, J. C. Bland, J. M. Johnson, C. S. G. Rogers, E. A. Stone, Harold C. Bird, Almon H. Fuller, Carlton T. Bishop, Clyde W. MacCornack, C. P. Disney, O. H. Ammann, Henry B. Seaman, Charles A. Mead, Herbert C. Keith, Philip George Lang, Jr., H. T. Welty, Clement C. Williams, Jonathan Jones, Glenn B. Woodruff, T. Kennard Thomson, C. R. Young, Edward Godfrey, A. F. Robinson, Lewis E. Moore, Charles B. Wing, R. O. Stewart, Henry S. Jacoby, Charles Evan Fowler, Charles F. Loweth, Ralph Modjeski, C. A. P. Turner, and D. B. Steinman.

Synopsis

For some time bridge engineers have recognized that the Cooper system of loading has ceased to provide a correct specification for the design of railway bridges, as it does not properly represent modern locomotive loadings. The specifying of different weights of Cooper engines for different lengths of span or for different members of the same span (as has been done) is, at best, a makeshift solution of the problem, and only serves to emphasize the need of a more consistent loading specification. In the successive attempts to keep pace with the continuous increase in railway loadings, the Cooper loading has been "strained beyond its elastic limit."

This paper presents a new loading system based on a study of the stress-producing effects of modern heavy locomotives.

The basic idea was to establish the maximum stresses producible at all sections of all spans by the heaviest existing locomotive loadings. The composite results showed a variation in equivalent Cooper ratings ranging from E-75 for the shorter spans to E-60 for the longer ones.

As the composite of the heaviest existing locomotive loadings is the best

* Presented at the meeting of September 6, 1922.

† Cons. Engr., New York City.

index of the directions in which the present average locomotive loadings may be expected to increase, it may fairly be taken as a standard or norm to which any new loading diagram proposed for the design of railway bridges ought to conform. The next step, therefore, was to devise a loading diagram or formula conforming as closely as practicable to this composite standard or norm. This paper offers three alternative solutions of this problem:

1.—A conventional wheel-load diagram, representing a locomotive of the Mallet type.

2.—A simplified loading diagram, consisting of a uniform load with three excess concentrations.

3.—A simple loading formula, giving directly the equivalent uniform load for any section of any span.

Each of these three alternative solutions has its distinctive advantages. The first represents the least departure from present methods, involving merely the substitution of a new and simpler wheel-load diagram for the one now in use.

The second solution offers a greatly simplified loading, which can be used expeditiously without tables or charts.

The third solution is the most scientific in principle and most convenient in application; tables, diagrams, and charts become endowed with simplicity and regularity; or, if tables and charts are not at hand, any maximum stress can be found by a simple operation on the ordinary slide-rule.

To expedite the application of the three proposed loading specifications, tables, charts, and isogonic diagrams of equivalent uniform loads are presented. Shear and moment graphs are also given, to show how closely the three proposed loadings conform to the composite standard for span lengths from 10 to 1 000 ft.

For each proposed loading system, the intensity corresponding to the composite standard is designated as Loading M-60. This loading can be increased or reduced in fixed ratios in the same manner as the Cooper system of loading. A rating chart and rating tables, included in the paper, permit quick comparison or conversion of E and M ratings for any member of any span.

Loading M-60 will give stresses for all spans very close to the maximum stresses producible by the heaviest existing locomotive loadings. It is about equivalent to Cooper's E-75 for short spans and to Cooper's E-60 for very long spans.

The adoption of Loading M-60 as a future loading specification instead of E-60 will insure that the short spans are strong enough for the heaviest locomotives and trains, without penalizing the longer spans with a 25% waste of metal.

The object of this paper is to direct attention to the inadequacy of the Cooper system to represent properly modern railway loadings; to submit, for the consideration of the Engineering Profession, three alternative forms of a new loading system based on the stress-producing effects of modern locomotives; and to secure, through discussion, a consensus of opinion as to which

of the three proposed forms should be recommended as a future loading specification.

Introduction

The Cooper system of loading was introduced into American bridge practice about 30 years ago. At that time, the heaviest loads to be provided for were properly represented by the Cooper E-40 loading, consisting of two consolidation locomotives with 40 000 lb. on each driver axle, followed by a uniform train load of 4 000 lb. per lin. ft.

Since then there has been a rapid and continuous augmentation of locomotive and train weights. To provide for this increase, the original Cooper E-40 loading was raised in successive specifications to the present values of E-60 and E-70 by applying fixed multiplying factors to the original loading diagram. This procedure could be applied indefinitely, without error, if the governing locomotive types and wheel spacings remained unchanged, and if the uniform train loads increased in the same proportion as the axle concentrations.

However, the old Consolidation locomotive has been superseded by new types of engines having different wheel groupings and weight distributions. The wheel diagrams of such modern locomotive types as the Mikado, Mallet, or Santa Fé, bear little resemblance to the Cooper engine diagram. Moreover, since the days of the E-40 loading, the axle concentrations have increased 100%, whereas the weight of train per linear foot has increased barely 50 per cent. It should be obvious, from these considerations, that a limit has been reached in the proportionate stretching of the old Cooper engine loading to make it fit modern loading conditions, and that the time has come to make a fresh start, with a system based on the existing locomotive and train loads.

Besides minor anomalies, the most serious indictment against the Cooper system is that bridges of uniform strength for present-day traffic cannot be obtained except by specifying different classes of the Cooper loading for different lengths of span and for different members of the same span. The maximum equivalent Cooper rating of the heaviest existing locomotives varies from E-75 for short spans to E-60 for long spans.

For the New York Central Lines, provision in part is made for this variation by specifying Cooper's E-70 for spans up to 200 ft. and E-65 for spans of more than 200 ft. This, however, does not provide for variations within a structure of given span.

In strengthening the Niagara Railway Arch Bridge, C. E. Fowler, M. Am. Soc. C. E., adopted E-70 for the floor system of the 550-ft. span; E-60 for the truss members of the same span; and E-70 for the truss members of the 115-ft. approach spans.

Such hybrid loading specifications, necessitated by the lack of correspondence between the Cooper system and modern train loads, have obvious disadvantages and are, at best, a makeshift solution of the problem. They serve only to emphasize the need of a new loading specification that will be more consistent with present-day traffic.

The Cooper loading does not correspond to modern locomotives, and its retention as a standard for design is unscientific. Its use produces designs that are unbalanced for present-day loadings. For instance, the Pennsylvania Railroad N-1-S locomotives are equivalent to Cooper's E-55 for some bridge members and to E-75 for others, a range of variation of 37 per cent. The Erie 2-10-2 locomotives are equivalent to Cooper's E-53 for some bridge members and to E-70 for others, a range of variation of 32 per cent. Other locomotives, compared by the writer, gave similar ranges of variation. In other words, structures designed for the arbitrary Cooper loading will not be of uniform strength for a given modern locomotive; some of the members in a span will be defective in strength in comparison with others, and some of the spans in a line will be disproportionately weak in comparison with others; and this difference in strength may amount to as much as 30 or 40 per cent.

An argument advanced for retaining the Cooper loading is that it affords a convenient standard for rating bridges; but the writer maintains that a standard that has become warped to such an extent as to yield results that fail to harmonize by 30 to 40%, has outlived its usefulness.

A new loading, in the form of a diagram, formula, or chart, should be developed for future specifications. This loading should satisfy the following requirements:

1.—It should give results that harmonize closely with the stresses producible by modern heavy locomotives.

2.—It should be heavy enough to provide for the probable future increase in loading during the desired life of the structure. Accordingly, it should be from 25 to 50% heavier than the average present loading (supposed to be represented by E-60).

3.—It should be presented in such form as will yield maximum facility of application and will discourage meaningless refinement of computation.

The foregoing considerations have prompted and guided the investigations embodied in this paper.

Composite of Heaviest Existing Locomotives

The first step toward establishing a new standard loading was to make an analysis of the stress-producing effects of the heaviest existing locomotives.

The most scientific way of studying, recording, and comparing the stress-producing characteristics of any specified loadings for all sections of all spans is on the basis of equivalent uniform loads. The equivalent uniform load for any section of any span represents the stress-producing value of a given loading, divested of all dimensional factors. Such a system of equivalent uniform loads was first published by the writer as a device for expediting the calculation of stresses for the Cooper loading. (See Plate IV.) The system is based on the principle that the equivalent uniform load for any stress depends only on the horizontal lengths, l_1 and l_2, of the two segments of the influence triangle for that stress. It is merely necessary to multiply this equivalent uniform load (taken from a table or chart) by a simple dimensional factor to obtain the maximum value of any stress.

Accordingly, the writer determined the equivalent uniform loads for the seven heaviest existing locomotives. The results are tabulated in Plate V. The locomotives included in this analysis and comparison are the following:

(1) The Erie Railroad Class P-1 (2-8-8-8-2).
(2) The Virginian Railway (2-10-10-2).
(3) Two Pennsylvania Railroad N-1-S engines (2-10-2).
(4) Two Erie engines (2-10-2).
(5) The Virginian Railway (2-8-8-2).
(6) The U. S. Standard (2-8-8-2) B.
(7) The Lake Shore and Michigan Southern Mallet (0-8-8-0).

Behind each of these locomotives is a uniform train load of 6 000 lb. per lin. ft., which fairly represents the heaviest continuous train loads. (An extra heavy coal car on the Virginian Railway weighs 5 940 lb. per ft. of track when loaded to capacity, and 6 380 lb. per ft. with 10% overload.)

The table given on Plate V covers a range of values of l_1 and l_2 extending from 10 to 1 000 ft. For each point in the table, there are recorded seven values of equivalent uniform loads corresponding, respectively, to the seven locomotives previously listed. The largest of the seven values in each block is indicated in bold-face type.

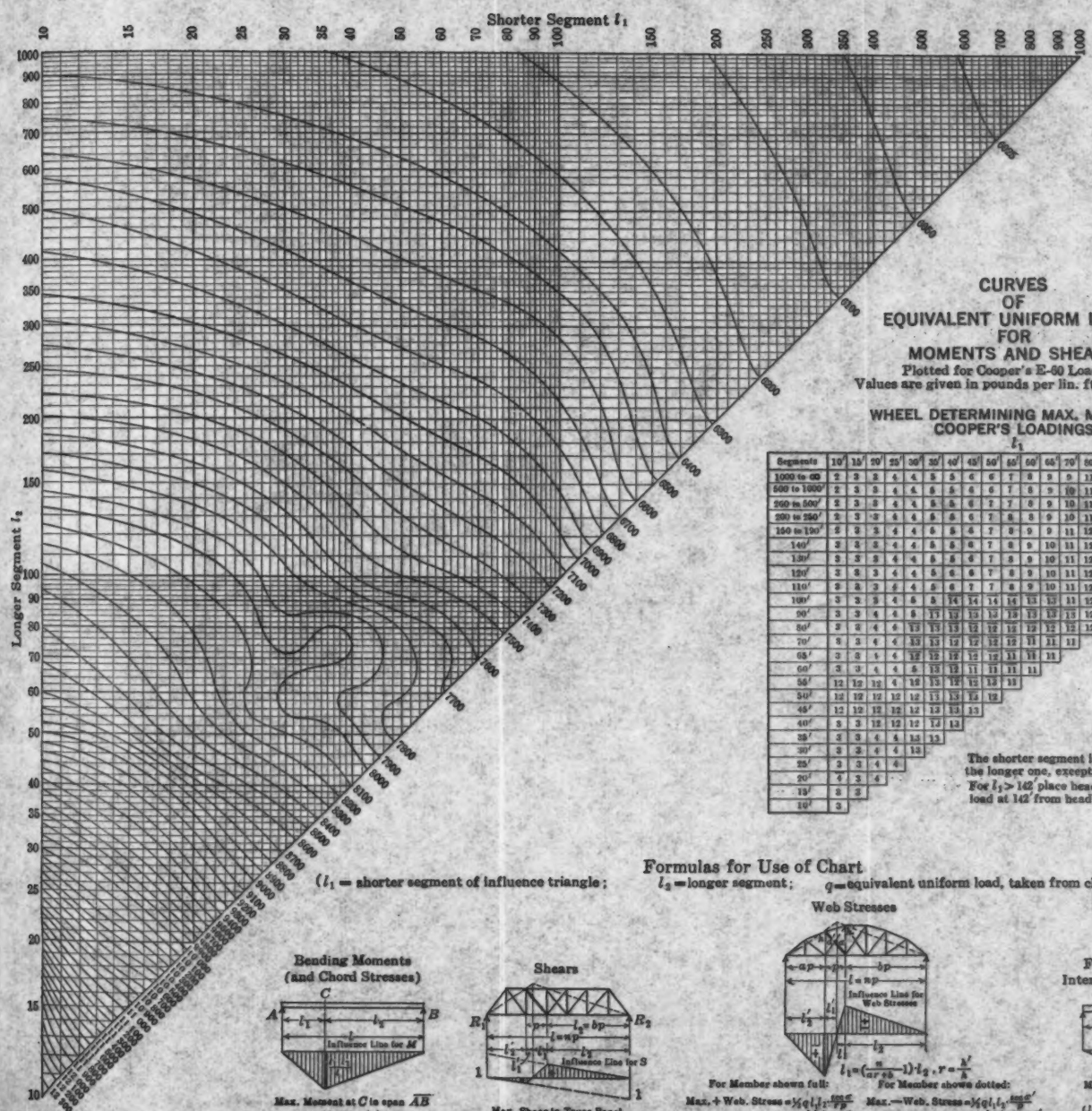
These maximum equivalent uniform loads, given in bold-face type on Plate V, represent a composite of the heaviest existing loadings. A table of these maximum values would constitute in itself an adequate loading specification to insure the safe and balanced design of all spans for the heaviest present-day locomotives.

The seven locomotives are listed in the general order of their stress-producing effects. There are, however, variations from this order; each locomotive has low points in the table, Plate V, where it is overlapped by one or more of the lighter locomotives. This fact constitutes an outstanding advantage of a composite loading, as it is free from arbitrary low points due to the individual peculiarities of a single locomotive.

It is found by inspection of the table, Plate V, that Locomotive No. 1 produces maximum stresses for all points above an indicated zigzag line corresponding approximately to a span of 150 ft. Between this line and a lower zigzag line, corresponding to a span of about 60 ft., Locomotive No. 2 produces maximum stresses. For all shorter spans, Locomotive No. 3 is the governing load. Therefore, Locomotives Nos. 1, 2, and 3 are sufficient to cover the entire field of existing locomotives as far as maximum stress-producing effect is concerned. A composite of these three engines would give stresses greater than any known loading for any section or member of any span.

In order to present in graphic form the composite stress-producing characteristics of the heaviest locomotives, the maximum values of the equivalent uniform loads were next platted on a chart, Plate VI. This diagram is of the same form as Plate IV for the Cooper loading, referred to previously. It possesses, however, striking advantages over the Cooper chart. The curves are more regular and the spacing is more even; the low points and kinks due to the idiosyncracies of the Cooper engine diagram are eliminated. The

PLATE IV.
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FOR RAILWAY BRIDGES.



CURVES OF EQUIVALENT UNIFORM LOADS FOR MOMENTS AND SHEARS

Plotted for Cooper's E-60 Loading
Values are given in pounds per lin. ft. per Track

WHEEL DETERMINING MAX. MOMENT COOPER'S LOADINGS
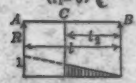
l_1

Segments	10′	15′	20′	25′	30′	35′	40′	45′	50′	55′	60′	65′	70′	80′	90′	100′	110′	120′	130′	140′
1000 to 60	2	3	3	4	4	5	5	6	6	7	8	9	9	11	12	13	14	15	17	19
500 to 1000′	2	3	3	4	4	5	5	6	6	7	8	9	10	11	12	13	14	16	17	18
260 to 500′	2	3	3	4	4	5	5	6	7	7	8	9	10	11	12	13	14	15	17	18
200 to 250′	2	3	3	4	4	5	5	6	7	8	8	9	10	11	12	13	14	15	17	18
150 to 120′	2	3	3	4	4	5	5	6	7	8	9	9	11	12	12	13	14	15	17	18
140′	2	3	3	4	4	5	5	6	7	8	9	10	11	12	12	13	14	15	17	18
130′	2	3	3	4	4	5	5	6	7	8	9	10	11	12	12	13	14	15	17	
120′	2	3	3	4	4	5	6	6	7	8	9	10	11	12	13	13	14	15		
110′	2	3	3	4	4	5	6	7	7	8	9	10	11	12	12	13	14			
100′	3	3	3	4	5	5	14	14	14	14	13	13	11	12	13	13				
90′	3	3	4	4	5	13	13	13	13	13	13	13	13	12	13					
80′	3	3	4	4	13	13	13	12	12	12	12	12	12	12						
70′	3	3	4	4	13	13	12	12	12	12	11	11	11							
65′	3	3	4	4	12	12	12	12	12	11	11	11								
60′	3	3	4	4	5	13	12	11	11	11	11									
55′	12	12	12	4	12	13	12	12	12	12										
50′	12	12	12	12	12	13	13	13	12											
45′	12	12	12	12	12	13	13	13												
40′	3	3	12	12	12	12	13													
35′	3	3	4	4	13	13														
30′	3	3	4	4	13															
25′	3	3	4	4																
20′	4	3	4																	
15′	3	3																		
10′	3																			

The shorter segment is ahead followed by the longer one, except wheel is overlined.
For $l_2 > 142'$ place head of uniform train load at 142′ from head of span.

Equivalent Uniform Loads for Shears and Reactions

($l_1 = 0$)



Max. Shear at C
$S = \frac{1}{2} q \frac{l_2^2}{l}$

Max. Reaction at A
($l_2 = l$)
$R = \frac{1}{2} q l$

l_2	q
1000	6418
975	6429
950	6440
925	6451
900	6463
875	6476
850	6490
825	6504
800	6518
775	6535
750	6553
725	6571
700	6591
675	6612
650	6634
625	6658
600	6682
575	6711
550	6741
525	6775
500	6812
475	6852
450	6895
425	6945
400	7000
375	7060
350	7130
325	7205
300	7295
275	7400
250	7520
225	7660
200	7830
175	8035
160	8185
150	8300
140	8420
130	8540
120	8680
110	8840
100	9000
95	9080
90	9140
85	9230
80	9315
75	9410
70	9470
65	9600
60	9860
55	10 160
50	10 460
45	10 900
40	11 310
35	11 930
30	12 650
25	13 625
20	15 000
15	16 000
10	18 000
5	24 000

Formulas for Use of Chart

(l_1 = shorter segment of influence triangle; l_2 = longer segment; q = equivalent uniform load, taken from chart for l_1 and l_2)

Bending Moments (and Chord Stresses)

Max. Moment at C in span $\overline{AB}$
$M = \frac{1}{2} q l_1 l_2$
(l_1 and l_2 = the segments of the span)

Shears

Max. Shear in Truss Panel
$S = \frac{1}{2} q \, l_1 l_2 \div p$
($l_1 = \frac{l_2}{n-1}$. $l_2 = bp$)

Web Stresses

$l_1 = \left(\frac{n}{ar+b} - 1\right) l_2$, $r = \frac{h'}{h}$

For Member shown full:
Max. + Web. Stress $= \frac{1}{2} q l_1 l_2 \frac{\sin\alpha}{rp}$
Max. − Web. Stress $= \frac{1}{2} q l_1 l_2 \frac{\sin\alpha}{p}$

For Member shown dotted:
Max. − Web. Stress $= \frac{1}{2} q l_1 l_2 \frac{\sin\alpha'}{p}$
Max. + Web. Stress $= \frac{1}{2} q \frac{l_1' l_2'}{l} \frac{\sin\alpha'}{p}$

Floorbeam Reactions Intermediate Pier Reactions



Max. Intermediate Reaction
$R = \frac{1}{2} q (l_1 + l_2)$
(l_1 and l_2 = the two adjoining spans.)

* Note:
In the above formula for S, the effect of wheel ① is neglected.
To correct this, subtract $20\,000\left(1 - \frac{l_2 - 1}{l_2}\right)$ from the above value of S

Short Segment I_1

	0	10	20	30	40	50	60	70	80	90	100	120	140	160	180	200	220	240	260	280	300	400	500	600	700	800	900	1000
1000	6409 6376 6353 6415 6388 6364 6350	6540 6500 6470 6520 6490 6460 6440	6580 6480 6440 6420 6340 6300 6280	6480 6460 6370 6360 6290 6240 6220	6440 6440 6360 6270 6220 6240 6150	6420 6400 6340 6280 6220 6210 6140	6400 6380 6290 6220 6210 6190 6170	6400 6360 6280 6190 6150 6170 6160	6380 6350 6280 6180 6140 6160 6060	6360 6320 6260 6160 6120 6140 6040	6340 6300 6240 6170 6100 6120 6030	6300 6260 6210 6150 6080 6100 6004	6220 6200 6210 6140 6110 6080 6028	6214 6160 6060 6110 6066 6070 6024	6186 6140 6130 6100 6058 6060 6022	6166 6130 6128 6080 6052 6055 6018	6148 6120 6120 6060 6046 6050 6017	6134 6110 6110 6060 6042 6045 6016	6122 6100 6100 6058 6038 6040 6014	6110 6090 6090 6070 6034 6040 6013	6102 6080 6080 6060 6032 6035 6012	6078 6060 6060 6050 6022 6030 6008	6052 6050 6050 6040 6018 6025 6006	6042 6040 6040 6030 6014 6020 6005	6034 6020 6030 6025 6010 6015 6004	6028 6020 6020 6020 6008 6010 6004	6024 6020 6020 6015 6007 6005 6004	6020 6010 6010 6010 6006 6004 6002
900	6413 6387 6364 6400 6419 6404 6395	6490 6480 6460 6440 6420 6400 6380	6540 6540 6480 6460 6420 6400 6340	6540 6540 6480 6420 6360 6360 6300	6500 6480 6440 6400 6360 6320 6260	6480 6460 6420 6380 6340 6300 6250	6460 6440 6420 6380 6340 6300 6250	6440 6400 6380 6340 6300 6260 6210	6420 6380 6360 6320 6280 6240 6200	6400 6360 6340 6300 6260 6220 6180	6380 6340 6320 6280 6240 6200 6160	6340 6300 6280 6240 6200 6160 6120	6340 6260 6220 6180 6140 6100 6060	6234 6160 6130 6072 6046 6060 6024	6204 6160 6120 6068 6040 6060 6022	6188 6140 6110 6058 6040 6020	6162 6130 6100 6060 6040 6050 6018	6146 6120 6100 6060 6040 6045 6016	6132 6100 6080 6058 6040 6040 6015	6120 6100 6070 6038 6040 6040 6014	6110 6080 6100 6060 6034 6035 6012	6076 6060 6060 6050 6024 6030 6008	6056 6040 6060 6040 6018 6025 6007	6044 6040 6040 6030 6014 6020 6006	6036 6030 6030 6025 6010 6015 6005	6030 6020 6020 6020 6010 6010 6004	6024 6010 6015 6015 6008 6006 6002	
800	6744 6734 6722 6714 6698 6679 6660	6600 6580 6560 6540 6520 6500 6480	6640 6580 6540 6500 6460 6420 6380	6620 6560 6500 6470 6420 6380 6340	6540 6500 6460 6420 6380 6340 6300	6520 6480 6440 6400 6360 6320 6280	6500 6460 6420 6380 6340 6300 6260	6480 6440 6400 6360 6320 6280 6240	6460 6420 6380 6340 6300 6260 6220	6440 6400 6360 6320 6280 6240 6200	6420 6380 6340 6300 6260 6220 6180	6380 6340 6300 6260 6220 6180 6140	6346 6280 6240 6190 6120 6110 6060	6258 6200 6240 6190 6094 6100 6064	6234 6180 6180 6140 6120 6070 6028	6198 6160 6160 6120 6062 6070 6022	6176 6140 6150 6110 6054 6065 6020	6158 6120 6140 6100 6060 6060 6018	6144 6110 6130 6090 6044 6055 6016	6132 6100 6120 6080 6040 6046 6015	6120 6090 6110 6060 6038 6040 6014	6092 6070 6060 6050 6028 6035 6010	6060 6060 6040 6040 6018 6025 6007	6048 6040 6030 6030 6014 6020 6006	6038 6030 6030 6025 6012 6015 6005	6030 6020 6020 6020 6010 6010 6004		
700	6840 6816 6796 6780 6764 6744 6717 6418	6700 6720 6680 6660 6640 6620 6600 6380	6720 6680 6640 6600 6560 6520 6480 6410	6640 6620 6560 6520 6480 6440 6400 6370	6620 6600 6540 6500 6460 6420 6380 6350	6580 6560 6520 6480 6440 6400 6360 6190	6560 6520 6480 6440 6400 6360 6320 6180	6540 6500 6460 6420 6380 6340 6300 6170	6520 6480 6440 6400 6360 6320 6280 6130	6500 6460 6420 6380 6340 6300 6260 6100	6480 6440 6400 6360 6320 6280 6240 6080	6460 6420 6380 6340 6300 6260 6220 6060	6400 6360 6320 6280 6240 6200 6160 6046	6320 6260 6220 6170 6130 6090 6052	6288 6240 6190 6170 6096 6100 6062	6256 6200 6180 6160 6078 6080 6028	6230 6180 6170 6130 6068 6080 6034	6196 6150 6160 6120 6060 6070 6022	6176 6140 6130 6100 6054 6060 6020	6160 6130 6140 6100 6060 6060 6018	6144 6120 6130 6090 6044 6055 6016	6132 6100 6120 6080 6040 6046 6015	6090 6070 6060 6050 6028 6035 6010	6066 6060 6040 6040 6018 6025 6007	6050 6040 6030 6030 6014 6020 6006	6040 6040 6030 6025 6012 6015 6005	6030 6020 6020 6020 6010 6010 6004		
600	6981 6962 6916 6876 6838 6801 6488	6860 6840 6780 6740 6700 6660 6620 6400	6840 6770 6720 6680 6640 6600 6560 6420	6800 6760 6700 6660 6620 6580 6540 6440	6760 6720 6680 6640 6600 6560 6520 6460	6780 6740 6700 6660 6620 6580 6540 6210	6680 6660 6620 6580 6540 6500 6460 6200	6640 6600 6560 6520 6480 6440 6400 6190	6680 6640 6600 6560 6520 6480 6440 6100	6560 6570 6570 6570 6570 6570 6570 6080	6500 6460 6420 6380 6340 6300 6260 6060	6440 6420 6340 6300 6260 6220 6180 6046	6386 6350 6340 6300 6260 6220 6180 6044	6326 6280 6300 6190 6192 6160 6110 6046	6282 6240 6270 6180 6140 6100 6060 6042	6248 6200 6190 6170 6130 6090 6050 6038	6226 6170 6180 6130 6088 6080 6040 6034	6196 6160 6170 6130 6062 6070 6022 6030	6178 6140 6160 6120 6056 6065 6020 6028	6162 6120 6140 6100 6050 6060 6018 6026	6146 6100 6120 6080 6046 6055 6016 6024	6190 6080 6060 6050 6030 6040 6012 6020	6072 6060 6060 6040 6020 6030 6008 6018	6056 6050 6040 6040 6018 6025 6006 6016		
500	7160 7122 7084 7038 6998 6958 6918	7060 7040 6980 6940 6900 6860 6820	6980 6960 6900 6860 6820 6780 6740	6920 6900 6840 6800 6760 6720 6680	6860 6840 6780 6740 6700 6660 6620	6820 6780 6740 6700 6660 6620 6580	6800 6760 6720 6680 6640 6600 6560	6780 6740 6700 6660 6620 6580 6540	6780 6740 6700 6660 6620 6580 6540	6660 6600 6610 6570 6530 6490 6450	6600 6540 6600 6560 6520 6480 6440	6540 6480 6520 6480 6440 6400 6360	6440 6380 6420 6380 6340 6300 6260	6376 6330 6340 6300 6260 6220 6180	6324 6280 6280 6240 6200 6160 6120	6282 6240 6240 6200 6160 6120 6080	6250 6210 6210 6170 6130 6090 6050	6224 6180 6180 6140 6100 6060 6020	6200 6160 6160 6120 6080 6040 6000	6182 6140 6140 6100 6060 6020 5980	6166 6120 6120 6080 6040 6000 5960	6110 6060 6060 6040 6020 6000 5980	6072 6060 6060 6040 6020 6000 5980	6050 6040 6040 6040 6020 6000 5980		

Longer Segment l_2

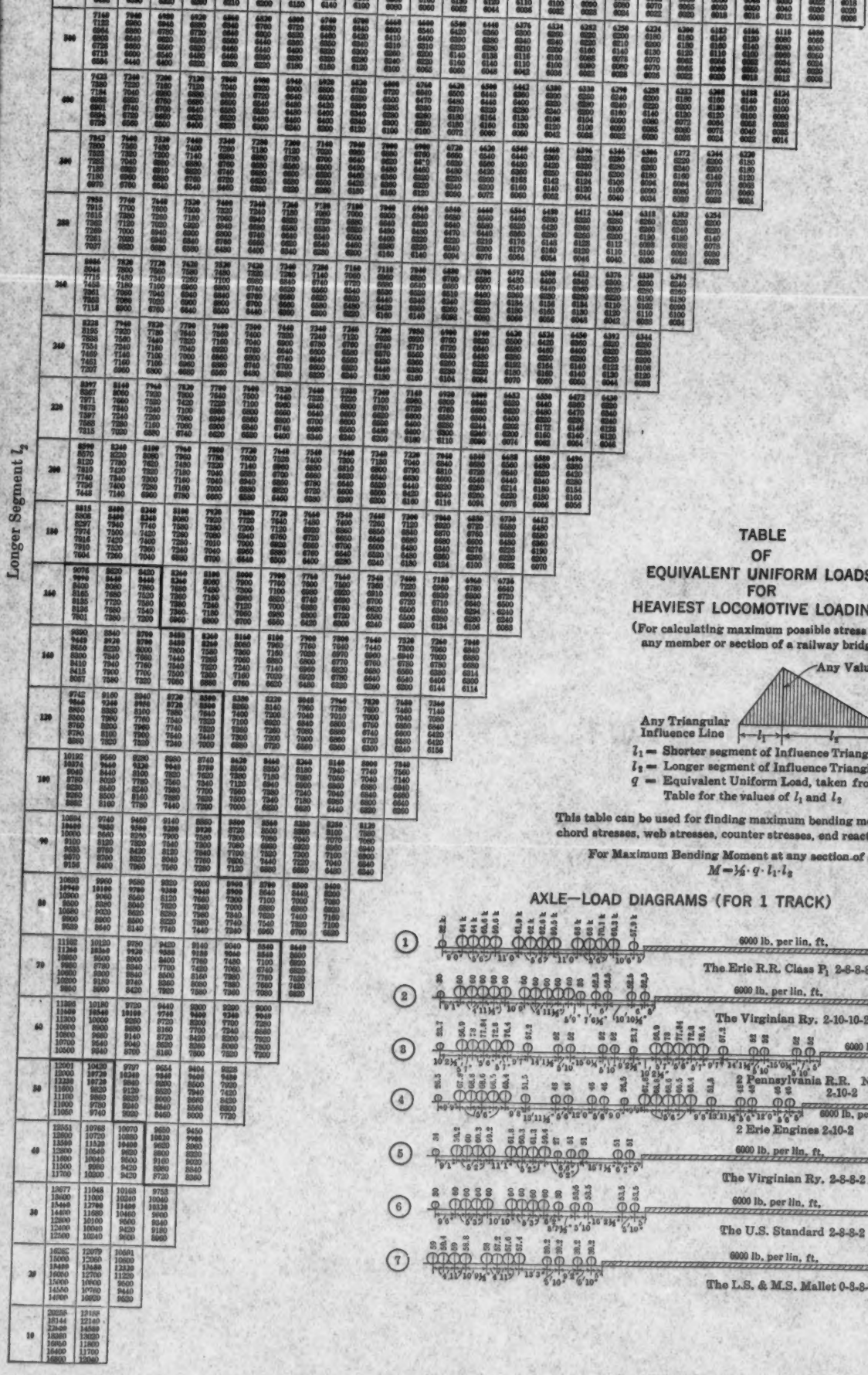


TABLE OF EQUIVALENT UNIFORM LOADS FOR HEAVIEST LOCOMOTIVE LOADINGS

(For calculating maximum possible stress in any member or section of a railway bridge)

l_1 = Shorter segment of Influence Triangle
l_2 = Longer segment of Influence Triangle
q = Equivalent Uniform Load, taken from Table for the values of l_1 and l_2

This table can be used for finding maximum bending moments, shears, chord stresses, web stresses, counter stresses, end reactions, etc.

For Maximum Bending Moment at any section of any span

$$M = \tfrac{1}{8} \cdot q \cdot l_1 \cdot l_2$$

AXLE—LOAD DIAGRAMS (FOR 1 TRACK)

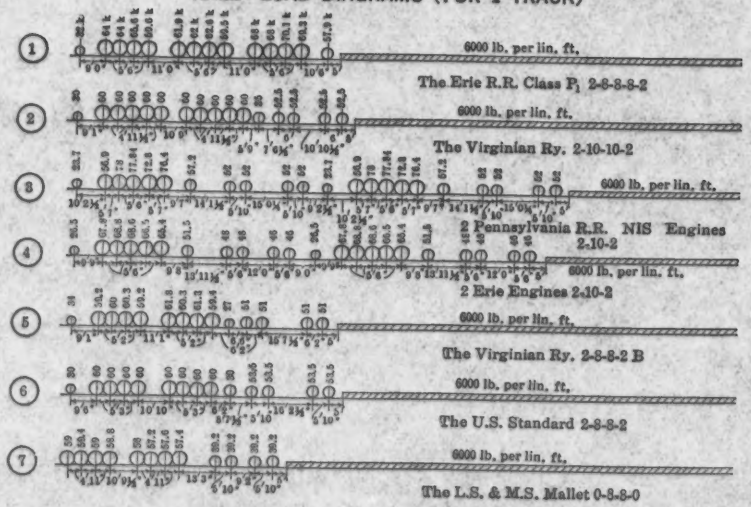


PLATE V.
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smoothness of the curves is an insurance against designs becoming unbalanced by future changes in locomotive wheel diagrams.

This chart, Plate VI, does not represent any single known engine but, instead, it represents a maximum composite of all existing engines and train loads, including the heaviest types. No known loading can produce a greater stress in any member than is given by this chart. Surely this fact alone ought to commend the chart as a standard for future loading specifications.

This chart, Plate VI, may be conceived as a plat of equivalent load curves, each enveloping the corresponding curves for all existing locomotives.

The author submits this chart, Plate VI, or its tabular form, Plate V, to the Profession as a standard or norm to which any new loading diagram proposed for the design of railway bridges ought to conform. If desired, this chart alone could be incorporated in bridge specifications as a simple and adequate prescription for all railway live load stresses. Bridges designed thereby would be safe, without waste, for the heaviest loadings thus far developed.

The loading, real or imaginary, represented by this chart, Plate VI, will hereinafter be referred to as the "Composite Standard Loading", and will be designated by the class symbol, M-60.

Plate VII presents a tabulation giving for each point the equivalent uniform load for the Composite Standard Loading and, also, the respective ratios thereto of the equivalent uniform loads for the seven heaviest existing locomotives.

This table, Plate VII, is presented, primarily, to show how little the Composite Standard Loading would be changed by the omission of any one of the seven loadings entering into it. For instance, if Locomotive No. 1 were omitted, the maximum reduction of stress would be only 3.6%; if Locomotive No. 2 were eliminated, the maximum stress reduction would be only 3.9%; if Locomotive No. 3 were omitted, there would be no stress reduction, except for very short spans; if any or all of the remaining locomotives Nos. 4 to 7, inclusive) were omitted, the Composite Standard Loading would be entirely unaffected.

As the omission of any of the heaviest locomotives effects so slight a reduction in the Composite Standard Loading, it is logical to expect that the addition of any new locomotive that may be developed in the near future will produce an equally slight augmentation in the Composite Standard Loading. In other words, the Composite Standard Loading provides for probable future development in load distribution as well as for present-day loading.

The tabulation of percentages in Plate VII is also useful in comparing the stresses given by the Composite Standard Loading and those producible by any one of the heavy locomotives listed. For instance, if Locomotive No. 7 (the Lake Shore and Michigan Southern Mallet) is the heaviest locomotive on a line, and if the spans do not exceed 300 ft., the tabulation on Plate VII shows that it will be sufficient to specify 90% of the Composite Standard Loading.

If the stresses for any bridge are calculated from the chart, Plate VI, for the Composite Standard Loading, the percentages tabulated in Plate VII can then be applied, if desired, to give the exact stresses producible by any specific heavy locomotive.

The tabulated percentages in Plate VII can also be used to show the safety, under any proposed engine, of a structure designed for any other engine in the list. Other uses of this table, Plate VII, will suggest themselves to the bridge engineer.

The table on Plate VIII gives the equivalent uniform load (per track) for the Cooper E-60 loading for any section in any span and, also, the corresponding Cooper ratings for the seven heaviest locomotive loadings, in order to permit direct comparison or conversion of stresses.

Aside from the general purpose of this paper, this table, Plate VIII, is useful for determining the overloading effects of any one of the heavy locomotives on a structure designed for a given Cooper loading.

The principal object in presenting this table, Plate VIII, however, is to show the disparity between the Cooper loading and the heavy locomotives now in use. Table 1 shows the extreme variations in equivalent Cooper ratings for the seven locomotives investigated.

TABLE 1.—EQUIVALENT COOPER RATINGS OF SEVEN HEAVIEST EXISTING LOCOMOTIVE LOADINGS.

Loading.	Minimum rating.	Maximum rating.	Range of variation.
1	E-60.1	E-72.0	20%
2	E-60.0	E-73.5	23 "
3	E-54.8	E-75.0	37 "
4	E-52.7	E-69.3	32 "
5	E-56.1	E-68.2	22 "
6	E-54.7	E-66.2	21 "
7	E-52.3	E-65.0	24 "

This attempt to apply the Cooper system as a measuring standard for rating existing railway loadings reveals the difficulties that have arisen. The Cooper system and modern loadings have become incommensurable. A given modern loading cannot be expressed as a given equivalent in the Cooper system, otherwise an error of 20 to 37% will be introduced. As a measuring or rating standard for present-day traffic, the Cooper system yields results that are radically inconsistent.

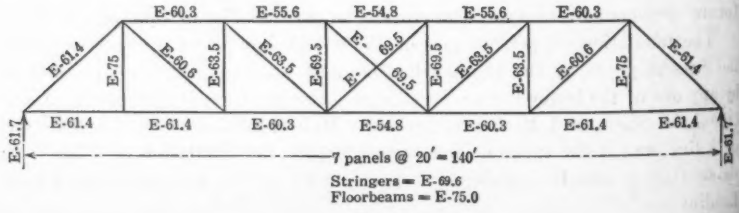
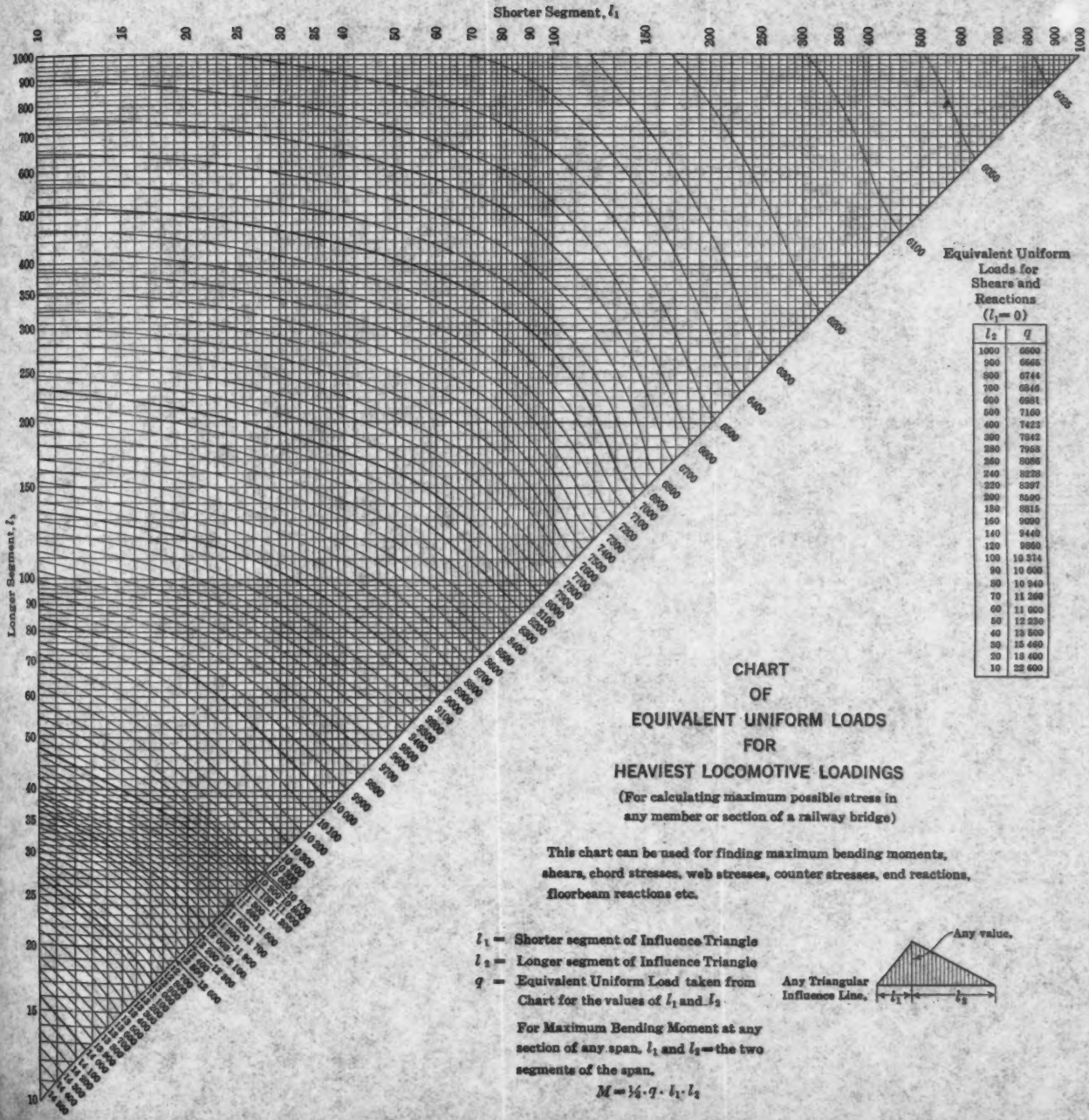


FIG. 1.

PLATE VI.
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Equivalent Uniform Loads for Shears and Reactions ($l_1 = 0$)

l_2	q
1000	6000
900	6665
800	6744
700	6846
600	6981
500	7160
400	7423
300	7842
280	7953
260	8086
240	8223
220	8397
200	8590
180	8815
160	9090
140	9440
120	9850
100	10 374
90	10 600
80	10 940
70	11 266
60	11 600
50	12 230
40	13 500
30	15 460
20	18 400
10	22 600

CHART OF EQUIVALENT UNIFORM LOADS FOR HEAVIEST LOCOMOTIVE LOADINGS

(For calculating maximum possible stress in any member or section of a railway bridge)

This chart can be used for finding maximum bending moments, shears, chord stresses, web stresses, counter stresses, end reactions, floorbeam reactions etc.

l_1 = Shorter segment of Influence Triangle
l_2 = Longer segment of Influence Triangle
q = Equivalent Uniform Load taken from Chart for the values of l_1 and l_2

Any Triangular Influence Line.

For Maximum Bending Moment at any section of any span, l_1 and l_2 = the two segments of the span.

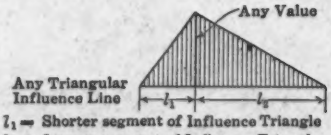
$$M = \tfrac{1}{8} \cdot q \cdot l_1 \cdot l_2$$

Shorter Segment z_1

	0	10	20	30	40	50	60	70	80	90	100	120	140	160	180	200	220	240	260	280	300	400	500	600	700	800	900	1000
1000	100.0 98.7 97.5 96.4 95.4

Longer Segment l_2

TABLE OF MAXIMUM AND RELATIVE EQUIVALENT UNIFORM LOADS FOR HEAVIEST LOCOMOTIVE LOADINGS



l_1 = Shorter segment of Influence Triangle
l_2 = Longer segment of Influence Triangle
q = Equivalent Uniform Load, taken from Table for the values of l_1 and l_2

This table can be used for finding maximum bending moments, shears, chord stresses, web stresses, counter stresses, end reactions, etc.

For Maximum Bending Moment at any section of any span
$M = \frac{1}{8} \cdot q \cdot l_1 \cdot l_2$

AXLE—LOAD DIAGRAMS (FOR 1 TRACK)

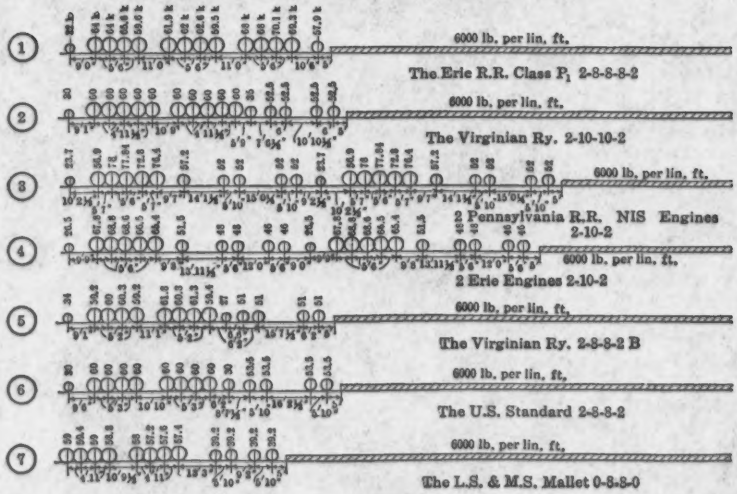


PLATE VII.
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STEINMAN ON
LOCOMOTIVE LOADINGS
FOR RAILWAY BRIDGES.

This disproportion between the Cooper system and actual modern loadings has made it almost prohibitively difficult for the railroads to "Cooperize" their bridges. The engineer of structures of one of the prominent lines recently told the writer that it had been given up as a hopeless task. When an attempt was made to fix the Cooper rating of a bridge designed for the road's actual equipment, one Cooper rating would be obtained for the stringers, another for the floor-beams, still another for the chord members, and a different rating again for the web members.

A simple illustration of this incommensurability of the Cooper system with modern loadings is given in Fig. 1, representing the Cooper rating diagram of a 140-ft. span designed for the Pennsylvania Railroad N-1-S engines. The ratings shown are taken directly from Plate VIII.

If the bridge is intended for the modern engine, but is designed according to the Cooper loading specification, there will be a relative overstressing or waste of metal in some of the parts, amounting to 37 per cent. Such variation or discrepancy appears to the writer to be a sufficient indictment against the Cooper system as a present-day specification.

The next step was to construct a graphic chart for rating the Composite Standard Loading (M-60) in terms of the Cooper system. This chart, presented on Plate IX, was obtained by plotting the maximum E-ratings taken from the tabulation on Plate VIII.

The kinks in this chart are not due to any irregularities in the Composite Standard; they represent the peculiarities of the Cooper loading diagram. The same irregularities are found in any chart or diagram relating to the Cooper loading, whereas a chart constructed for the Composite Standard (such as Plate VI) is comparatively smooth and free from disturbing irregularities.

This chart, Plate IX, can be used as a rating chart for comparing or converting the stresses from the Cooper loading and the stresses from the Composite Loading (M-60) proposed herein as a standard. Thus, at the middle of a 40-ft. span ($l_1 = 20$, $l_2 = 20$), the proposed loading is equivalent to Cooper's E-75. At the quarter-points of a 400-ft. span ($l_1 = 100$, $l_2 = 300$), the proposed loading is equivalent to Cooper's E-63.4.

Table 2 shows the Cooper ratings of the proposed loading (M-60), for various spans, obtained from Plate IX and the corresponding table, Plate VIII.

It is also shown in Table 2 that the proposed Composite Loading (M-60) is equivalent to about Cooper's E-75 at short spans and to Cooper's E-60 at long spans. In other words, the Cooper rating of the proposed loading (M-60) gradually diminishes from E-75 to E-60 as the length of span is increased.

This variation shows that bridges of different span length in the same line will not be of uniform strength under modern loading, if they are designed for a uniform class of the Cooper loading. If designed for E-60, the shorter spans will be overstressed 25%; if designed for E-75, the longer spans will have a 25% waste of metal. Under these conditions, a single loading that will result in bridges of uniform strength for all spans and for all members in any span, should be preferred to the Cooper system. If designed

for the proposed M-60 loading, all members of all spans will be of uniform strength under modern loading conditions, and any member of any span will be strong enough, without waste, to carry the heaviest existing locomotive loads.

TABLE 2.—EQUIVALENT COOPER RATINGS OF THE PROPOSED COMPOSITE LOADING (M-60).

Spans.	E-rating for center moments.	E-rating for end shears.
10	E-72	E-72
20	E-73	E-74
30	E-74	E-73
40	E-75	E-72
50	E-73	E-70
60	E-72	E-71
70	E-72	E-71
80	E-73	E-71
90	E-73	E-70
100	E-73	E-69
120	E-71	E-68
140	E-69	E-67
160	E-68	E-67
180	E-67	E-66
200	E-66	E-66
250	E-65	E-65
300	E-63	E-65
400	E-62	E-64
500	E-61	E-63
600	E-61	E-63
700	E-61	E-62
800	E-61	E-62
900	E-60	E-62
1 000	E-60	E-62

Having constructed, as a substitute for the Cooper system, a composite of the heaviest existing locomotives in the form of a table (Plate V) and a chart (Plate VI) of maximum equivalent uniform loads, the problem of devising a more condensed and more tangible form of representing this new loading system will be discussed. In the following pages, the writer presents three alternative solutions of this problem:

1.—A proposed engine diagram, consisting of a simple conventional system of axle concentrations, followed by a uniform train load.

2.—A simplified loading diagram, consisting of a uniform load with a floating group of three excess concentrations.

3.—A loading formula, giving directly the equivalent uniform load for maximum stress at any section of any span.

These three alternative methods of specifying the proposed loading system will be presented and discussed in turn.

PROPOSED ENGINE LOADING

The engine loading diagram devised to represent the composite of the heaviest existing locomotives is shown on Figs. 2, 3, 4, 5, 6, and 7, and Plate X.

Of necessity, a locomotive of the Mallet type was adopted. (Five of the seven heaviest locomotives, as shown on Plate V, are of the Mallet type.)

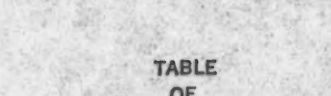
Five axles to each group of drivers were found to be necessary. (Three of the four heaviest locomotives have Decapod grouping.)

Shorter Segment l_1

Longer Segment l_2

	0	10	20	30	40	50	60	70	80	90	100	120	140	160	180	200	220	240	260	280	300	400	500	600	700	800	900	1000
1000	61.7 61.6 59.9 59.6 58.8	61.7 61.2 59.6 59.3 58.6	61.5 61.3 60.0 59.5 58.7	61.5 61.4 60.1 59.7 58.4	61.5 61.5 60.2 59.5 58.6	61.4 61.3 60.2 59.5 58.4	61.4 61.4 60.2 59.7 58.6	61.6 61.2 60.4 59.5 58.5	61.5 61.3 60.2 59.6 58.4	61.7 61.2 60.2 59.7 58.5	61.6 61.2 60.2 59.5 58.5	61.3 61.0 59.9 59.3 58.5	60.7 60.5 59.9 59.3 58.5	60.8 60.5 59.8 59.3 58.5	60.7 60.5 59.8 59.3 58.5	60.7 60.3 59.9 59.3 58.5	60.6 60.3 59.9 59.3 58.5	60.5 60.3 59.9 59.3 58.5	60.4 60.2 59.8 59.3 58.5	60.4 60.2 59.9 59.3 58.5	60.4 60.2 59.9 59.3 58.5	60.3 60.2 59.9 59.3 58.5	60.2 60.2 59.9 59.3 58.5	60.2 60.2 59.9 59.3 58.5	60.2 60.2 59.9 59.3 58.5	60.2 60.2 59.9 59.3 58.5	60.1 60.1 59.9 59.3 58.5	60.1 60.1 59.9 59.3 58.5
900	61.8 61.6 60.0 59.6 58.8	61.8 61.3 59.8 59.4 58.6	61.6 61.4 60.0 59.5 58.7	61.8 61.5 60.2 59.7 58.4	61.7 61.6 60.2 59.5 58.6	61.7 61.4 60.2 59.5 58.4	61.7 61.5 60.2 59.7 58.6	61.8 61.4 60.4 59.5 58.5	61.8 61.5 60.2 59.6 58.4	61.8 61.3 60.2 59.7 58.5	61.8 61.3 60.2 59.5 58.5	61.6 61.1 59.9 59.3 58.5	60.8 60.7 59.9 59.3 58.5	60.9 60.6 59.8 59.3 58.5	60.8 60.6 59.8 59.3 58.5	60.8 60.5 59.9 59.3 58.5	60.6 60.5 59.9 59.3 58.5	60.6 60.5 59.9 59.3 58.5	60.5 60.4 59.8 59.3 58.5	60.4 60.3 59.9 59.3 58.5	60.4 60.2 59.9 59.3 58.5	60.3 60.2 59.9 59.3 58.5	60.2 60.2 59.9 59.3 58.5	60.2 60.2 59.9 59.3 58.5	60.2 60.2 59.9 59.3 58.5	60.2 60.2 59.9 59.3 58.5	60.1 60.1 59.9 59.3 58.5	60.1 60.1 59.9 59.3 58.5
800	62.1 61.8 60.0 59.6 58.8	62.1 61.5 59.9 59.4 58.6	61.9 61.4 60.0 59.5 58.7	62.2 61.8 60.2 59.7 58.4	62.0 61.8 60.2 59.5 58.6	62.0 61.6 60.2 59.5 58.4	62.0 61.8 60.2 59.7 58.6	62.1 61.5 60.4 59.5 58.5	62.1 61.5 60.2 59.6 58.4	62.1 61.5 60.2 59.7 58.5	62.0 61.7 60.2 59.5 58.5	61.8 61.0 59.9 59.3 58.5	61.0 60.9 59.9 59.3 58.5	61.0 60.8 59.8 59.3 58.5	61.0 60.8 59.8 59.3 58.5	61.0 60.7 59.9 59.3 58.5	60.8 60.6 59.9 59.3 58.5	60.8 60.5 59.9 59.3 58.5	60.7 60.5 59.8 59.3 58.5	60.6 60.4 59.9 59.3 58.5	60.5 60.4 59.9 59.3 58.5	60.4 60.3 59.9 59.3 58.5	60.3 60.2 59.9 59.3 58.5	60.2 60.2 59.9 59.3 58.5	60.2 60.2 59.9 59.3 58.5	60.2 60.2 59.9 59.3 58.5	60.1 60.1 59.9 59.3 58.5	60.1 60.1 59.9 59.3 58.5
700	62.2 62.0 60.0 59.6 58.8	62.2 61.8 59.9 59.4 58.6	62.0 61.7 60.0 59.5 58.7	62.2 62.0 60.2 59.7 58.4	62.2 62.0 60.2 59.5 58.6	62.0 61.8 60.2 59.5 58.4	62.0 62.0 60.2 59.7 58.6	62.1 61.8 60.4 59.5 58.5	62.1 61.8 60.2 59.6 58.4	62.0 61.8 60.2 59.7 58.5	62.0 62.0 60.2 59.5 58.5	61.8 61.2 59.9 59.3 58.5	61.2 61.0 59.9 59.3 58.5	61.1 61.0 59.8 59.3 58.5	61.0 61.0 59.8 59.3 58.5	61.0 60.8 59.9 59.3 58.5	60.8 60.7 59.9 59.3 58.5	60.8 60.6 59.9 59.3 58.5	60.7 60.6 59.8 59.3 58.5	60.6 60.4 59.9 59.3 58.5	60.5 60.4 59.9 59.3 58.5	60.4 60.3 59.9 59.3 58.5	60.3 60.2 59.9 59.3 58.5	60.3 60.2 59.9 59.3 58.5	60.2 60.2 59.9 59.3 58.5	60.2 60.2 59.9 59.3 58.5	60.2 60.2 59.9 59.3 58.5	
600	62.7 62.5 60.0 59.6 58.8	62.5 62.2 59.9 59.4 58.6	62.2 62.0 60.0 59.5 58.7	62.2 62.2 60.2 59.7 58.4	62.2 62.2 60.2 59.5 58.6	62.2 62.0 60.2 59.5 58.4	62.2 62.2 60.2 59.7 58.6	62.4 62.0 60.4 59.5 58.5	62.4 62.0 60.2 59.6 58.4	62.3 61.8 60.2 59.7 58.5	62.3 61.8 60.2 59.5 58.5	62.1 61.7 59.9 59.3 58.5	61.8 61.6 59.9 59.3 58.5	61.5 61.5 59.8 59.3 58.5	61.4 61.2 59.8 59.3 58.5	61.5 61.0 59.9 59.3 58.5	61.1 61.0 59.9 59.3 58.5	61.0 60.8 59.9 59.3 58.5	60.8 60.7 59.8 59.3 58.5	60.7 60.6 59.9 59.3 58.5	60.7 60.4 59.9 59.3 58.5	60.6 60.4 59.9 59.3 58.5	60.4 60.3 59.9 59.3 58.5	60.4 60.2 59.9 59.3 58.5	60.3 60.2 59.9 59.3 58.5	60.3 60.2 59.9 59.3 58.5	60.2 60.2 59.9 59.3 58.5	
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400	63.6 63.3 60.0 59.6 58.8	63.3 63.1 59.9 59.4 58.6	63.2 63.1 60.0 59.5 58.7	63.4 63.3 60.2 59.7 58.4	63.3 63.3 60.2 59.5 58.6	63.3 63.2 60.2 59.5 58.4	63.3 63.3 60.2 59.7 58.6	63.4 63.3 60.4 59.5 58.5	63.4 63.3 60.2 59.6 58.4	63.4 63.2 60.2 59.7 58.5	63.3 63.2 60.2 59.5 58.5	63.2 63.1 59.9 59.3 58.5	63.1 63.0 59.9 59.3 58.5	63.0 63.0 59.8 59.3 58.5	63.0 62.9 59.8 59.3 58.5	62.9 62.8 59.9 59.3 58.5	62.8 62.7 59.9 59.3 58.5	62.7 62.6 59.8 59.3 58.5	62.6 62.5 59.9 59.3 58.5	62.5 62.4 59.9 59.3 58.5	62.4 62.3 59.9 59.3 58.5	62.3 62.2 59.9 59.3 58.5	62.2 62.2 59.9 59.3 58.5	62.2 62.2 59.9 59.3 58.5	62.2 62.2 59.9 59.3 58.5	62.2 62.2 59.9 59.3 58.5	
300	64.0 64.1 60.0 59.6 58.8	64.0 64.4 59.9 59.4 58.6	64.0 64.3 60.0 59.5 58.7	64.2 64.4 60.2 59.7 58.4	64.2 64.4 60.2 59.5 58.6	64.2 64.3 60.2 59.5 58.4	64.2 64.4 60.2 59.7 58.6	64.4 64.4 60.4 59.5 58.5	64.4 64.4 60.2 59.6 58.4	64.3 64.3 60.2 59.7 58.5	64.3 64.2 60.2 59.5 58.5	64.1 64.0 59.9 59.3 58.5	63.9 63.9 59.9 59.3 58.5	63.8 63.8 59.8 59.3 58.5	63.8 63.7 59.8 59.3 58.5	63.7 63.6 59.9 59.3 58.5	63.6 63.5 59.9 59.3 58.5	63.5 63.4 59.8 59.3 58.5	63.4 63.3 59.9 59.3 58.5	63.3 63.2 59.9 59.3 58.5	63.2 63.2 59.9 59.3 58.5	63.2 63.1 59.9 59.3 58.5	63.1 63.1 59.9 59.3 58.5	63.1 63.1 59.9 59.3 58.5	63.1 63.1 59.9 59.3 58.5	63.1 63.1 59.9 59.3 58.5	
250	64.7 64.5 61.8 59.6 58.8	64.8 64.6 59.9 59.4 58.6	64.8 64.6 60.0 59.5 58.7	65.0 64.8 60.2 59.7 58.4	64.8 64.8 60.2 59.5 58.6	64.8 64.6 60.2 59.5 58.4	64.8 64.8 60.2 59.7 58.6	64.8 64.6 60.4 59.5 58.5	64.8 64.6 60.2 59.6 58.4	64.8 64.6 60.2 59.7 58.5	64.6 64.5 60.2 59.5 58.5	64.2 64.1 59.9 59.3 58.5	63.8 63.8 59.9 59.3 58.5	63.6 63.6 59.8 59.3 58.5	63.5 63.4 59.8 59.3 58.5	63.4 63.3 59.9 59.3 58.5	63.2 63.1 59.9 59.3 58.5	63.1 63.0 59.8 59.3 58.5	62.9 62.8 59.9 59.3 58.5	62.8 62.7 59.9 59.3 58.5	62.7 62.6 59.9 59.3 58.5	62.6 62.5 59.9 59.3 58.5	62.5 62.5 59.9 59.3 58.5	62.5 62.5 59.9 59.3 58.5	62.5 62.5 59.9 59.3 58.5	62.5 62.5 59.9 59.3 58.5	
200	65.5 64.8 61.8 59.6 58.8	65.1 65.0 59.9 59.4 58.6	65.0 64.7 60.0 59.5 58.7	65.3 64.7 60.2 59.7 58.4	65.3 64.8 60.2 59.5 58.6	65.4 64.6 60.2 59.5 58.4	65.4 64.8 60.2 59.7 58.6	64.9 64.2 60.4 59.5 58.5	64.9 64.1 60.2 59.6 58.4	64.3 63.2 60.2 59.7 58.5	64.5 62.8 60.2 59.5 58.5	63.2 62.9 59.9 59.3 58.5	62.3 61.8 59.9 59.3 58.5	62.2 61.6 59.8 59.3 58.5	62.4 61.9 59.8 59.3 58.5	62.2 61.2 59.9 59.3 58.5	61.8 61.0 59.9 59.3 58.5	61.6 60.9 59.8 59.3 58.5	61.4 60.8 59.9 59.3 58.5								
150	66.2 64.1 62.2 59.8 59.4 57.1	66.0 64.0 62.4 59.6 59.1 57.0	66.0 64.0 62.4 59.6 59.1 57.0	66.2 64.0 62.4 59.6 59.1 57.0	66.2 64.0 62.4 59.6 59.1 57.0	66.2 64.0 62.4 59.6 59.1 57.0	66.2 64.0 62.4 59.6 59.1 57.0	66.1 64.0 62.4 59.6 59.1 57.0	66.0 64.0 62.4 59.6 59.1 57.0	65.5 64.3 62.4 59.6 59.1 57.0	65.2 63.9 62.4 59.6 59.1 57.0	64.5 62.9 62.4 59.6 59.1 57.0	63.8 62.6 62.4 59.6 59.1 57.0	63.5 62.3 62.4 59.6 59.1 57.0	63.0 61.9 62.4 59.6 59.1 57.0	62.4 61.7 62.4 59.6 59.1 57.0	61.2 61.2 62.4 59.6 59.1 57.0	63.1 59.6 59.1 57.0	59.5 59.1 57.0								
100	66.5 66.6 61.5 59.8 59.6 57.2	66.5 66.6 62.0 59.2 59.6 56.9	66.5 66.7 62.0 59.2 59.6 56.8	66.6 66.6 62.0 59.2 59.6 56.8	67.1 66.8 62.0 59.2 59.6 56.8	67.1 66.8 62.0 59.2 59.6 56.8	67.2 66.9 62.0 59.2 59.6 56.8	66.5 66.2 62.0 59.2 59.6 56.8	66.9 66.8 62.0 59.2 59.6 56.8	66.2 66.3 62.0 59.2 59.6 56.8	66.3 66.3 62.0 59.2 59.6 56.8	64.8 63.9 62.0 59.2 59.6 56.8	64.3 63.7 62.0 59.2 59.6 56.8	63.6 62.1 62.0 59.2 59.6 56.8	62.6 61.8 62.0 59.2 59.6 56.8												
90	67.0 67.3 61.7 59.8 59.6 57.2	66.8 67.4 62.0 59.2 59.6 56.9	67.3 67.4 62.0 59.2 59.6 56.8	67.3 67.4 62.0 59.2 59.6 56.8	67.3 67.5 62.0 59.2 59.6 56.8	67.5 67.5 62.0 59.2 59.6 56.8	67.5 67.6 62.0 59.2 59.6 56.8	67.5 67.7 62.0 59.2 59.6 56.8	67.5 67.7 62.0 59.2 59.6 56.8	67.1 67.4 62.0 59.2 59.6 56.8	66.1 66.7 62.0 59.2 59.6 56.8	65.0 65.2 62.0 59.2 59.6 56.8	64.6 63.6 62.0 59.2 59.6 56.8	64.0 62.6 62.0 59.2 59.6 56.8														
80	67.3 68.1 61.8 59.8 59.6 57.2	67.5 68.2 62.0 59.2 59.6 56.9	67.8 68.1 62.0 59.2 59.6 56.8	68.1 68.1 62.0 59.2 59.6 56.8	68.4 68.4 62.0 59.2 59.6 56.8	68.5 68.5 62.0 59.2 59.6 56.8	68.6 68.7 62.0 59.2 59.6 56.8	68.6 68.6 62.0 59.2 59.6 56.8	68.6 68.6 62.0 59.2 59.6 56.8	68.0 68.3 62.0 59.2 59.6 56.8	67.1 66.3 62.0 59.2 59.6 56.8	66.6 65.0 62.0 59.2 59.6 56.8	66.0 64.2 62.0 59.2 59.6 56.8	65.8 63.9 62.0 59.2 59.6 56.8	64.9 63.0 62.0 59.2 59.6 56.8												
70	68.0 69.1 61.9 59.8 59.6 57.2	69.0 70.5 62.0 59.2 59.6 56.9	68.9 69.4 62.0 59.2 59.6 56.8	69.2 69.6 62.0 59.2 59.6 56.8	69.2 69.7 62.0 59.2 59.6 56.8	69.2 69.7 62.0 59.2 59.6 56.8	68.6 67.7 62.0 59.2 59.6 56.8	68.6 66.2 62.0 59.2 59.6 56.8	68.9 66.3 62.0 59.2 59.6 56.8	68.5 66.3 62.0 59.2 59.6 56.8	65.5 64.3 62.0 59.2 59.6 56.8	65.0 63.6 62.0 59.2 59.6 56.8	64.9 63.3 62.0 59.2 59.6 56.8														
60	69.4 69.6 62.0 59.8 59.6 57.2	69.4 70.4 62.0 59.2 59.6 56.9	69.6 69.8 62.0 59.2 59.6 56.8	69.4 70.0 62.0 59.2 59.6 56.8	69.3 69.9 62.0 59.2 59.6 56.8	68.9 68.5 62.0 59.2 59.6 56.8	68.5 67.2 62.0 59.2 59.6 56.8	68.1 66.2 62.0 59.2 59.6 56.8	67.5 66.3 62.0 59.2 59.6 56.8	67.5 65.5 62.0 59.2 59.6 56.8	66.9 64.9 62.0 59.2 59.6 56.8																
50	70.1 70.5 62.0 59.8 59.6 57.2	70.2 71.2 62.0 59.2 59.6 56.9	70.2 71.0 62.0 59.2 59.6 56.8	69.7 70.2 62.0 59.2 59.6 56.8	68.5 68.8 62.0 59.2 59.6 56.8	68.8 68.2 62.0 59.2 59.6 56.8	68.2 67.6 62.0 59.2 59.6 56.8	67.6 67.1 62.0 59.2 59.6 56.8	68.5 66.9 62.0 59.2 59.6 57.1																		
40	70.4 71.3 62.0 59.8 59.6 57.2	70.5 72.0 62.0 59.2 59.6 56.9	71.0 71.0 62.0 59.2 59.6 56.8	70.5 69.5 62.0 59.2 59.6 56.8	69.0 69.4 62.0 59.2 59.6 56.8	69.5 69.7 62.0 59.2 59.6 56.8	68.2 68.2 62.0 59.2 59.6 56.8	68.3 68.0 62.0 59.2 59.6 56.8																		
30	69.9 71.0 62.0 59.8 59.6 57.2	70.7 72.2 62.0 59.2 59.6 56.9	70.6 72.5 62.0 59.2 59.6 56.8	71.2 72.5 62.0 59.2 59.6 56.8	70.7 71.8 62.0 59.2 59.6 56.8	70.7 71.0 62.0 59.2 59.6 56.8	71.0 70.9 62.0 59.2 59.6 56.8																		
20	68.9 69.9 62.0 59.8 59.6 57.2	69.6 71.5 62.0 59.2 59.6 56.9	69.6 72.1 62.0 59.2 59.6 56.8	71.6 73.2 62.0 59.2 59.6 56.8	71.8 72.9 62.0 59.2 59.6 56.8	72.0 73.1 62.0 59.2 59.6 56.8																		

TABLE OF EQUIVALENT COOPER RATINGS FOR HEAVIEST LOCOMOTIVE LOADINGS



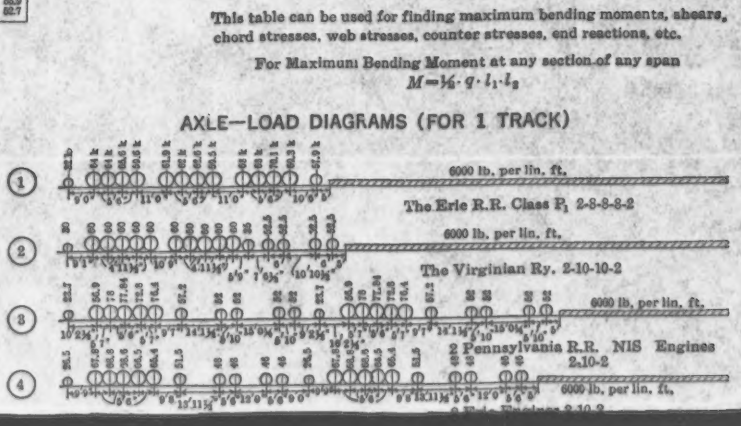
l_1 = Shorter segment of Influence Triangle
l_2 = Longer segment of Influence Triangle
q = Equivalent Uniform Load, taken from Table for the values of l_1 and l_2

This table can be used for finding maximum bending moments, shears, chord stresses, web stresses, counter stresses, end reactions, etc.

For Maximum Bending Moment at any section of any span

$$M = \tfrac{1}{8} \cdot q \cdot l_1 \cdot l_2$$

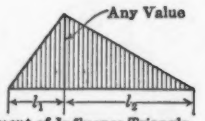
AXLE—LOAD DIAGRAMS (FOR 1 TRACK)



Longer Segment l_2

	240	220	200	180	160	140	120	100	90	80	70
240	65.5 64.8 64.0 59.9 59.2 58.1 57.1	65.5 65.2 62.5 59.8 59.2 58.1 57.1	65.8 65.7 65.2 59.3 58.3 57.1	66.2 65.1 62.2 59.8 59.4 58.4 57.1	66.5 66.6 61.5 59.8 59.6 59.6 57.2	67.0 67.3 61.7 60.2 59.7 60.0 57.4	67.3 68.1 61.2 58.3 60.5 60.7 57.7	68.0 69.1 60.2 58.3 61.2 61.7 59.0	69.4 69.6 65.6 59.7 62.5 62.0 60.0	70.1 70.5 70.2 68.2 63.7 57.1	70.4 71.3 65.0 64.9 63.1 59.7 57.1
220	9674	7460	7535	7994	8185	8430	8668	9000	9440	9915	10,400
	66.0 65.3 62.0 59.5 58.7 56.9	65.3 65.0 61.7 59.4 59.1 58.8 56.5	65.7 65.5 61.7 59.4 59.2 59.0 56.5	66.0 66.0 61.9 59.4 59.2 58.9 56.3	66.5 66.6 62.3 59.2 59.5 59.2 56.9	67.3 67.4 62.0 59.2 59.7 59.6 56.6	68.1 68.2 61.8 58.9 60.5 60.7 57.6	68.9 69.0 60.9 57.9 61.8 61.9 58.8	69.6 70.4 61.7 57.9 62.5 62.0 59.5	70.2 71.2 63.8 59.1 63.3 62.9 56.6	70.5 71.8 65.0 64.9 63.1 59.7 57.1
200	7535	7460	7535	7994	8185	8430	8668	9000	9440	9915	10,400
	66.8 65.7 62.2 59.3 58.3 57.1	66.0 65.3 62.0 59.5 58.7 56.9	65.7 65.5 61.7 59.4 59.2 59.0 56.5	66.0 66.0 61.9 59.4 59.2 58.9 56.3	66.2 66.0 62.0 59.3 59.5 59.1 56.1	67.3 67.3 62.0 59.2 60.0 59.6 56.3	68.1 68.1 61.8 58.8 60.5 60.7 57.6	68.9 69.4 60.9 57.9 61.8 61.9 58.8	69.6 70.4 61.7 57.9 62.5 62.0 59.5	70.2 71.2 63.8 59.1 63.3 62.9 56.6	70.5 71.8 65.0 64.9 63.1 59.7 57.1
180	7994	7994	8185	8430	8668	9000	9440	9915	10,400	10,900	11,400
	66.2 65.1 62.2 59.8 59.4 58.4 57.1	66.0 66.0 62.0 59.0 58.3 58.1 57.0	66.0 66.0 61.9 59.4 59.2 58.9 56.3	66.2 66.0 61.9 59.3 59.5 59.1 56.1	66.2 66.2 62.0 59.3 59.6 59.3 56.3	67.3 67.3 61.8 59.2 60.0 59.5 56.1	68.4 68.4 61.4 58.8 60.4 59.8 56.3	69.2 68.1 59.2 57.2 60.6 60.7 57.0	69.2 69.4 59.2 57.2 60.7 61.2 57.0	68.6 67.6 58.2 57.0 60.7 61.2 57.0	67.6 66.8 57.2 56.4 59.2 58.7 56.2
160	8185	8185	8430	8668	9000	9440	9915	10,400	10,900	11,400	11,900
	66.5 66.6 61.5 59.8 59.6 59.6 57.2	65.5 66.6 62.3 59.2 59.5 59.2 56.9	66.5 66.7 62.0 59.2 59.8 59.5 56.8	66.6 66.6 62.0 59.3 59.6 59.5 56.1	67.1 66.8 62.0 59.3 60.0 59.5 56.3	67.1 66.7 61.2 58.5 59.6 59.2 56.2	67.2 65.9 60.3 57.9 59.5 58.9 55.7	65.5 65.2 59.8 57.8 59.0 58.3 55.1	65.9 64.6 59.4 57.7 58.2 57.7 54.2	65.2 63.6 57.9 57.2 56.9 57.1 54.0	64.8 63.3 60.6 57.1 57.1 54.4
140	8430	8430	8668	9000	9440	9915	10,400	10,900	11,400	11,900	12,400
	67.0 67.3 61.7 60.2 59.7 60.0 57.4	66.8 67.4 62.0 59.2 59.7 59.7 58.1	67.3 67.3 61.9 59.2 60.0 59.6 56.6	67.3 67.3 62.0 59.2 60.0 59.6 56.3	67.3 67.3 61.8 59.2 59.7 59.5 56.1	67.5 66.8 60.8 58.3 59.8 59.2 55.8	67.8 66.7 60.0 57.7 58.8 58.7 55.4	68.7 65.6 59.3 57.4 58.8 58.5 54.8	66.1 64.7 59.1 57.4 57.9 57.5 53.6	65.0 63.2 59.2 57.5 56.8 56.4 53.2	65.0 63.1 60.0 56.7 56.7 56.6 53.6	64.6 62.8 62.3 60.3 56.7 56.9 54.7	64.0 62.2 62.6 60.7 57.4 57.2 55.6
120	8668	8668	9000	9440	9915	10,400	10,900	11,400	11,900	12,400	12,900
	67.3 68.1 61.2 58.3 60.5 60.7 57.7	67.5 68.2 61.8 58.9 60.5 59.7 57.6	67.8 68.1 61.4 58.3 60.3 59.8 57.0	68.1 68.1 61.8 58.3 60.5 59.5 56.6	68.4 68.4 61.4 58.3 60.4 59.8 56.3	68.5 67.5 60.5 57.7 60.2 59.8 55.2	68.0 67.4 59.7 57.2 60.2 59.6 55.7	67.1 66.3 58.6 57.0 59.0 58.3 54.7	66.6 65.0 58.5 57.5 57.3 57.2 53.2	66.0 64.2 59.2 57.1 57.0 56.8 53.2	65.8 63.9 60.3 56.8 56.4 56.5 53.5	64.9 63.0 62.5 60.4 56.7 56.7 54.4
100	9000	9000	9440	9915	10,400	10,900	11,400	11,900	12,400	12,900	13,400
	68.0 69.1 60.2 58.3 61.2 61.7 59.0	69.0 70.5 60.9 57.9 61.8 61.9 58.8	69.0 69.3 60.8 57.8 61.8 61.2 57.5	68.9 69.4 60.0 57.8 61.2 60.5 57.1	69.2 68.5 59.2 57.6 60.9 60.3 57.0	69.2 68.1 59.2 57.2 60.6 60.7 56.1	68.6 67.6 58.2 57.0 59.2 58.7 55.2	67.6 66.8 57.0 56.7 58.6 58.2 54.2	66.9 65.3 56.8 56.2 58.2 57.8 53.2	65.5 64.3 58.5 57.2 56.5 56.5 52.4	66.0 63.6 60.1 56.4 56.1 56.2 53.7	64.3 62.7
90	9440	9440	9915	10,400	10,900	11,400	11,900	12,400	12,900	13,400	13,900
	69.4 69.6 65.6 59.7 62.5 62.0 60.0	69.4 70.4 61.7 57.9 62.5 62.0 59.5	69.6 69.9 60.8 57.6 61.9 61.1 58.6	69.4 69.9 60.0 57.3 61.7 61.0 57.4	69.3 69.0 59.2 57.3 61.2 60.4 56.1	68.9 68.3 57.2 55.4 59.5 59.2 55.8	68.1 67.6 55.2 53.2 59.2 58.3 54.8	67.8 66.8 54.7 53.2 58.9 58.2 53.7	67.2 65.5 53.5 53.2 58.5 57.7 52.7	66.9 64.9 58.3 57.1 56.8 56.5 52.3		
80	9915	9915	10,400	10,900	11,400	11,900	12,400	12,900	13,400	13,900	14,400
	70.1 70.5 70.2 68.2 63.7 57.1	70.2 71.2 63.8 59.1 63.3 62.9 56.6	70.2 71.0 62.7 59.0 63.0 62.2 56.5	69.7 70.2 60.7 57.0 62.0 61.5 57.9	68.8 68.8 58.5 56.3 60.7 60.0 56.5	68.8 68.2 56.4 55.4 60.6 59.9 56.1	68.2 67.6 55.6 54.9 59.7 59.1 54.5	67.6 67.1 54.7 54.6 59.3 58.8 53.7	67.5 65.8 53.2 53.2 58.9 57.5 52.7	65.5 65.3 56.9 55.8 57.5 57.1 52.4		
70	10,400	10,400	10,900	11,400	11,900	12,400	12,900	13,400	13,900	14,400	14,900
	70.4 71.3 65.0 63.1 63.7 59.7 57.1	70.5 72.0 65.3 64.9 63.1 59.7 57.1	71.0 71.9 64.9 63.0 62.9 59.6 56.5	70.3 70.9 64.0 62.0 62.0 59.5 56.3	69.0 69.4 60.7 57.4 60.7 60.0 56.1	69.5 69.7 59.2 56.2 60.6 59.9 56.1	69.2 69.2 57.2 55.2 59.7 59.1 54.5	68.3 68.0 56.2 54.2 59.3 58.8 53.7				
60	10,900	10,900	11,400	11,900	12,400	12,900	13,400	13,900	14,400	14,900	15,400
	71.0 71.9 65.0 63.1 63.7 59.7 57.1	71.0 72.0 65.3 64.9 63.1 59.7 57.1	71.0 71.9 64.9 63.0 62.9 59.6 56.5	70.3 70.9 64.0 62.0 62.0 59.5 56.3	69.0 69.4 60.7 57.4 60.7 60.0 56.1	69.5 69.7 59.2 56.2 60.6 59.9 56.1	69.2 69.2 57.2 55.2 59.7 59.1 54.5	68.3 68.0 56.2 54.2 59.3 58.8 53.7				

TABLE
OF
EQUIVALENT COOPER RATINGS
FOR
HEAVIEST LOCOMOTIVE LOADINGS



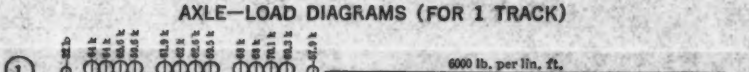
l_1 = Shorter segment of Influence Triangle
l_2 = Longer segment of Influence Triangle
q = Equivalent Uniform Load, taken from Table for the values of l_1 and l_2

This table can be used for finding maximum bending moments, shears, chord stresses, web stresses, counter stresses, end reactions, etc.

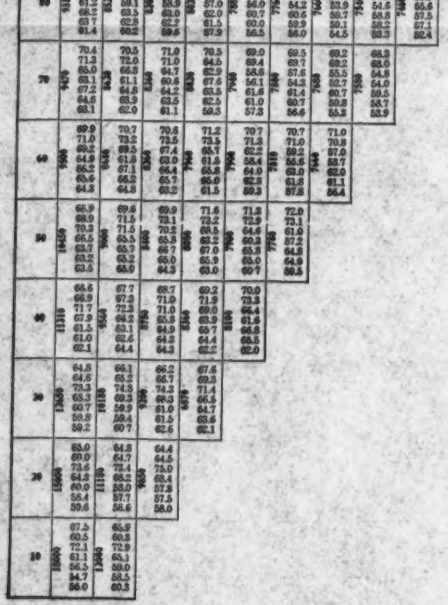
For Maximum Bending Moment at any section of any span

$$M = \tfrac{1}{8} \cdot q \cdot l_1 \cdot l_2$$

AXLE—LOAD DIAGRAMS (FOR 1 TRACK)



The Erie R.R. Class P. 2-8-8-8-2



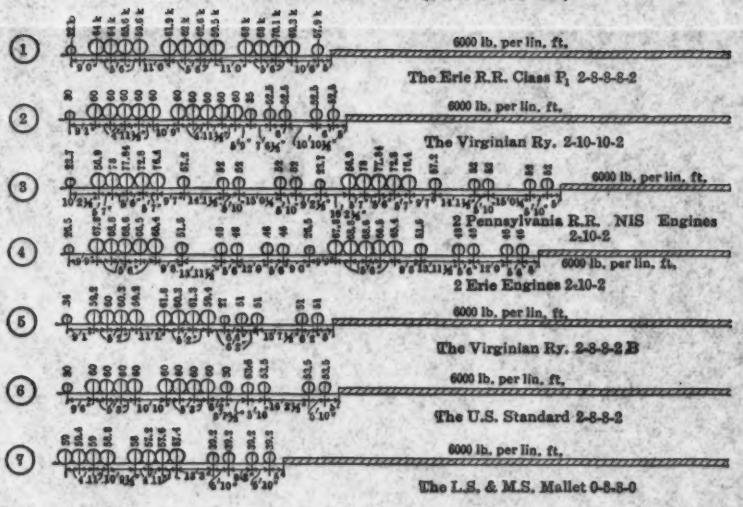


PLATE VIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXVI, No. 1520.
STEINMAN ON
LOCOMOTIVE LOADINGS
FOR RAILWAY BRIDGES.

PLATE IX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXVI, No. 1520.
STEINMAN ON
LOCOMOTIVE LOADINGS
FOR RAILWAY BRIDGES.

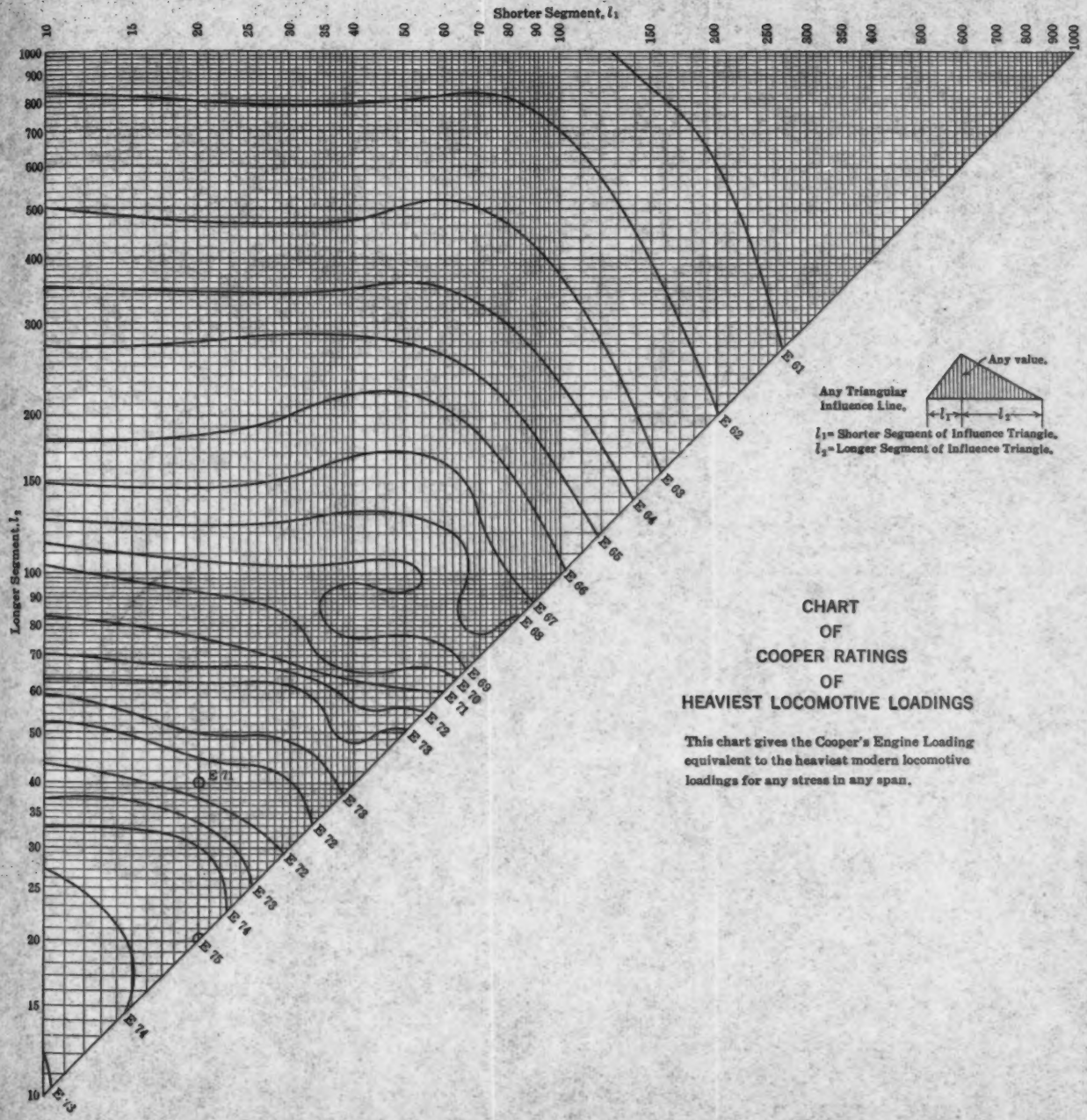


CHART OF COOPER RATINGS OF HEAVIEST LOCOMOTIVE LOADINGS

This chart gives the Cooper's Engine Loading equivalent to the heaviest modern locomotive loadings for any stress in any span.

This makes a total of ten driving axles, the same as in the heaviest locomotive loadings, Nos. 2, 3, and 4. (In Nos. 5, 6, and 7, there are only eight driving axles. In No. 1, there are twelve driving axles).

In order to obtain the same stresses for small spans as are produced by the heavy group of driver concentrations of Locomotive No. 3 (Pennsylvania Railroad N-1-S), it was found necessary to make one group of axle loads in the proposed engine diagram, 75 000 lb. each. Each of the axle loads in the other group were made 60 000 lb., in order to keep down the stresses for longer spans to the values produced by Locomotives Nos. 1 and 2.

The spacing of the 60 000-lb. axles was made 5 ft., as this is the closest spacing of such axles in any existing loading. The spacing of the 75 000-lb. axles was made 5½ ft., for the same reason. (In Locomotives Nos. 2, 5, 6, and 7, the 60 000-lb. axles are spaced approximately 5 ft. apart; and in Locomotives Nos. 1, 3, and 4, the heavier axles are spaced 5½ ft. apart.) The heavier group of axles was placed in the rear, following the precedent of Locomotive No. 1.

A pair of pilot wheels was added, as required by the stresses, in accordance with the precedent fixed by the six heaviest locomotives. The trailing wheels and tender, however, are considered as being absorbed in the uniform train load which follows the locomotive. As modern cars have concentrated loads exceeding those of the trailing wheels and tender, it is inconsistent refinement to represent the one by concentrations and the other by a uniform load.

The proposed engine diagram is thus reduced to a conventional 2-10-10 grouping of wheels, followed by a uniform train load of 6 000 lb. per lin. ft.

Although the engine diagram is really determined by the conditions previously set forth, the writer arrived at the same diagram by a synthetic method which closely fixed each axle load and each wheel spacing.

In constructing a chart of equivalent uniform loads, one begins with a wheel diagram and proceeds to find the corresponding equivalent uniform loads. In the present case, the problem is reversed: The equivalent uniform loads are listed for all sections of all spans in the table, Plate V, and the chart, Plate VI; it is required to construct a wheel diagram to fit these equivalent uniform loads throughout the entire range of spans and sections.

This problem is solved by starting with very short spans and determining the necessary maximum concentration. Then, by successively lengthening the span considered, successive additional axle loads are determined in magnitude and relative position, until a point is reached where the further annexing of concentrations can be dispensed with and a uniform load substituted. This, in brief, is the method that was followed by the writer, except that minor modifications of the loads and spacings were introduced in order to secure greater uniformity and simplicity.

After the engine diagram was thus determined, it was found to give results very close to those obtained with the composite of the heaviest locomotives, except for slight variations due to the rounding off of the figures adopted.

In order to facilitate the use of the proposed engine diagram and at the same time to test its conformity with the Composite Standard, the equivalent uniform loads have been computed and are presented in tabular form in Fig. 2.

Shorter Segment l_1

Longer Segment l_2	0	10	20	30	40	50	60	70	80	90	100	120	140	160	180	200	220	240	260	280	300	400	500	600	700	800	900	1000
1000	6589	6520	6500	6475	6450	6420	6400	6380	6360	6340	6310	6263	6220	6191	6167	6148	6132	6118	6108	6099	6091	6063	6047	6037	6030	6025	6021	6018
900	6652	6585	6560	6530	6500	6460	6445	6425	6400	6380	6350	6289	6243	6208	6182	6161	6144	6130	6117	6107	6098	6068	6051	6039	6032	6026	6022	
800	6731	6650	6620	6590	6560	6520	6490	6470	6440	6420	6390	6321	6269	6230	6201	6177	6158	6142	6129	6117	6107	6076	6054	6042	6034	6028		
700	6830	6740	6710	6670	6625	6590	6560	6540	6500	6470	6440	6360	6301	6257	6224	6197	6175	6157	6148	6129	6118	6080	6059	6045	6036			
600	6962	6855	6820	6770	6720	6670	6640	6610	6570	6540	6500	6410	6342	6291	6252	6221	6196	6176	6158	6144	6131	6089	6064	6049				
500	7143	7010	6960	6900	6840	6780	6750	6690	6670	6630	6580	6475	6395	6335	6289	6253	6223	6199	6179	6162	6148	6098	6071					
400	7406	7240	7160	7100	7025	6950	6920	6860	6810	6760	6700	6567	6469	6395	6339	6295	6260	6231	6206	6186	6169	6111						
300	7826	7600	7530	7410	7300	7170	7160	7100	7020	6940	6830	6704	6575	6481	6410	6354	6318	6274	6243	6218	6197							
280	7943	7675	7590	7490	7380	7270	7220	7140	7080	7000	6880	6738	6602	6503	6429	6369	6322	6284	6258	6226								
260	8072	7790	7700	7570	7450	7350	7300	7220	7150	7060	6950	6777	6632	6526	6447	6385	6335	6295	6262									
240	8222	7930	7800	7680	7550	7440	7380	7310	7220	7120	7020	6820	6665	6554	6468	6403	6350	6307										
220	8395	8080	7930	7790	7660	7540	7460	7400	7300	7200	7090	6868	6703	6583	6492	6421	6365											
200	8594	8250	8075	7920	7780	7650	7580	7500	7390	7280	7160	6923	6744	6615	6517	6443												
180	8825	8430	8230	8080	7910	7780	7710	7610	7500	7370	7240	6983	6790	6650	6547													
160	9110	8630	8430	8270	8080	7940	7860	7750	7620	7480	7340	7053	6844	6692														
140	9440	8930	8660	8470	8260	8120	8040	7900	7750	7600	7450	7132	6903															
120	9840	9250	8940	8680	8430	8340	8250	8100	7900	7760	7580	7230																
100	10 320	9610	9250	9000	8730	8600	8500	8390	8140	7940	7730																	
90	10 580	9810	9420	9150	8850	8750	8660	8480	8260	8050																		
80	11 180	10 000	9590	9300	9050	8940	8320	8640	8390																			
70	11 840	10 170	9730	9450	9250	9150	9020	8820																				
60	12 380	10 470	9860	9600	9500	9400	9230																					
50	12 840	11 100	10 250	9780	9700	9660																						
40	13 825	11 900	10 900	10 160	9930																							
30	15 820	12 920	11 650	10 550																								
20	17 630	13 990	12 685																									
10	21 750	14 250																										



l_1 = Shorter Segment of Influence Triangle.
l_2 = Longer Segment of Influence Triangle.
q = Equivalent Uniform Load taken from Chart for the values of l_1 and l_2 Values given in pounds per linear foot per track.

This table can be used for figuring the maximum stress in any member or section of any span. It gives stresses approximately equal to the maximum stresses producible by the heaviest existing locomotive loadings.

For Maximum Bending Moment at any section of any span, l_1 and l_2 = the two segments of the span.

$$M = \tfrac{1}{2} \cdot q \cdot l_1 \cdot l_2$$

For Maximum End Reaction or End Shear, $l_1 = 0$ and l_2 = the span.

$$V = \tfrac{1}{2} \cdot q \cdot l_2$$

For Maximum Shear at any section of a span, $l_1 = 0$, l_2 = the longer segment of the span and l = the span.

$$V = \tfrac{1}{2} \cdot q \cdot l_2 \cdot \frac{l_2}{l}$$

For Maximum Floorbeam Reaction or stress in Hanger, l_1 and l_2 = the adjoining panel lengths

$$R = \tfrac{1}{2} \cdot q \cdot (l_1 + l_2)$$

TABLE OF EQUIVALENT UNIFORM LOADS (PER TRACK) FOR DESIGNING RAILWAY BRIDGES

Based on Proposed Engine Loading M-60

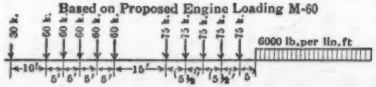


FIG. 2.

This table, Fig. 2, gives the equivalent uniform loads of the proposed engine loading (M-60) for all sections of all spans. It is found to check closely the maximum values in the similar table, Plate V, constructed for the heaviest existing locomotive loadings.

Simple methods of using this table for calculating maximum bending moments, shears, and reactions are indicated. More complete instructions for calculating maximum stresses of all kinds from equivalent uniform loads will be found on Plate IV.

Fig. 3 is a conventional tabulation of load positions giving maximum stress under the proposed engine loading. It gives the wheel to be placed at the section for any combination of values of l_1 and l_2. The equivalent uniform loads tabulated in Fig. 2 correspond to the load positions given in this table (Fig. 3).

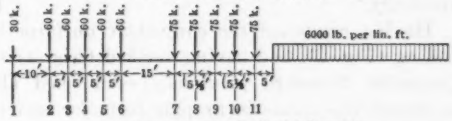
The table (Fig. 3) shows greater regularity than the corresponding table for the Cooper loading (Plate IV). For the proposed engine loading, this table (Fig. 3) is hardly needed, as the wheel diagram is so simple and regular that load positions for maximum stress at any section can be decided by inspection. It is seldom necessary to try more than one load position for any maximum stress.

Shorter Segment l_1

Longer Segment l_2	0	10	20	30	40	50	60	70	80	90	100	120	140	160	180	200	220	240	260	280	300	400	500	600	700	800	900	1000
1000	2	3	3	4	5	6	7	8	9	9	10																	
900	2	3	3	4	5	6	7	8	9	9	10																	
800	2	3	3	4	5	6	7	8	9	9	10																	
700	2	3	3	4	5	6	7	8	9	9	10																	
600	2	3	3	4	5	6	7	8	9	9	10																	
500	2	3	3	4	5	6	7	8	9	9	10																	
400	2	3	3	4	5	6	7	8	9	9	10																	
300	2	3	3	4	5	6	7	8	9	10	10																	
280	2	3	3	4	5	6	7	8	9	10	10																	
260	2	3	3	4	5	7	7	8	9	10	10																	
240	2	3	3	4	5	7	7	8	9	10	10																	
220	2	3	3	4	5	7	7	8	9	10	10																	
200	2	3	3	4	6	7	7	8	9	10	10																	
180	2	3	3	5	6	7	8	8	9	10	10																	
160	2	3	3	5	6	7	8	8	9	10	10																	
140	2	3	4	5	6	7	8	8	9	10	10																	
120	2	3	4	5	6	7	8	8	9	10	10																	
100	2	3	4	5	6	8	8	9	9	10	10																	
90	2	3	4	5	6	8	8	9	9	10																		
80	$\overline{11}$	3	4	5	7	8	8	9	9																			
70	$\overline{11}$	3	5	5	7	8	8	9																				
60	$\overline{11}$	8	5	5	7	8	8																					
50	$\overline{11}$	8	9	8	7	8																						
40	$\overline{11}$	8	9	8	7																							
30	7	8	9	8																								
20	7	8	9																									
10	7	8																										

For $l_1 > 117\frac{1}{2}$ ft., place head of uniform train load at $117\frac{1}{2}$ ft. from head of span.

WHEEL DETERMINING MAXIMUM STRESS
PROPOSED ENGINE LOADING M-60



The shorter segment is ahead followed by the longer one, except wheel is overlined.

FIG. 3.

When l_1 increases beyond the value of 117.5 ft., the head of the uniform train load becomes fixed at that distance from the head of the span. Con-

sequently, for all values of l_1 exceeding 117.5 ft., the equivalent uniform load, in kips per foot, is expressible by Equation (1):

$$q = 6 + \frac{35\,320}{l_1\,(l_1 + l_2)} \quad \ldots\ldots (1)$$

For mid-span moments, $l_1 = l_2$, and Equation (1) reduces to:

$$q = 6 + \frac{17\,660}{l_2^2} \quad \ldots\ldots (2)$$

For end shears in spans of 90 ft. or more, the proposed engine loading yields an equivalent uniform load expressible by Equation (3):

$$q = 6 + \frac{606}{l_2} - \frac{17\,440}{l_2^2} \quad \ldots\ldots (3)$$

The use of Equations (1), (2), and (3), and other similar formulas, greatly simplified the computation of the equivalent uniform loads for the table on Fig 2. Moreover, these formulas represent continuous functions of l_1 and l_2 over wide areas, and, therefore, contribute to smoothness and regularity in charts or diagrams of equivalent uniform loads constructed for the proposed engine loading (Plate X and Figs. 5 and 6).

The remarkably close correspondence between the proposed engine loading (M-60) and the composite of the heaviest existing locomotives is shown in the comparative rating table, Fig. 4. This table gives, for each point, the percentage ratio of the equivalent uniform load for the proposed engine loading to the corresponding equivalent uniform load of the Composite Standard. The percentages tabulated are simply the ratios of the values in Fig. 2 to the corresponding maximum values in Plate V.

The tabulated ratios are all close to 100%; with few exceptions the variation is less than 1 per cent. At a few points, confined to very short spans, some larger variations are found, with extreme values of — 4 and + 6 per cent. These variations could have been eliminated by making one or two of the heavy axle loads in the proposed engine loading slightly more than 75 000 lb. and the adjacent axle loads correspondingly less; but the writer considered it wiser to sacrifice such hair-splitting refinement for the sake of the greater simplicity and convenience resulting from the use of five equal axle loads of 75 000 lb. The writer doubts whether any closer agreement with the Composite Standard could be obtained by the use of any other engine diagram of equal simplicity.

Having computed the equivalent uniform loads for the proposed engine loading (Fig. 2), and having verified (Fig. 4) their close agreement with the Composite Standard previously established (Plate V), the next step was to present the values in graphic form for easier application. This is done in Plate X which is a chart of equivalent uniform loads for the proposed engine loading and on which are notes that explain its use.

This chart, Plate X, agrees closely with the corresponding chart for the Composite Standard (Plate VI). The maximum variation between corresponding curves on the two charts is, except for a few unimportant points, only 1 or 2 per cent.

PERCENTAGE RATINGS
OF
PROPOSED ENGINE LOADING—M-60
IN TERMS OF
HEAVIEST LOCOMOTIVE LOADINGS

Shorter Segment, l_1

Larger Segment, l_2	0	10	20	30	40	50	60	70	80	90	100	120	140	160	180	200	220	240	260	280	300	400	500	600	700	800	900	1000
1000	99.9	99.7	100.0	100.0	100.2	100.0	100.0	99.7	99.7	99.7	99.5	99.4	100.0	99.5	99.6	99.7	99.7	99.5	99.7	99.8	99.9	99.8	100.0	99.9	100.0	100.0	100.0	100.0
900	99.8	99.8	100.0	99.9	100.0	99.7	99.8	99.8	99.7	99.7	99.5	99.2	100.0	99.6	99.8	99.7	99.7	99.7	99.7	99.8	100.0	99.9	100.0	100.0	100.0	100.0	100.0	
800	99.8	99.5	99.7	99.5	100.0	99.7	99.5	99.5	99.4	99.7	99.9	99.4	99.5	99.5	99.6	99.5	99.7	99.8	99.7	99.7	99.9	99.9	99.9	99.9	100.0	100.0		
700	99.8	99.7	99.9	100.2	100.0	100.1	100.0	100.0	100.0	99.8	99.7	99.4	99.7	99.5	99.5	99.5	99.8	99.7	99.9	99.7	99.7	99.9	100.0	100.0	100.0			
600	99.8	100.0	99.7	99.6	99.5	99.5	99.4	99.5	99.6	99.7	100.0	99.5	99.4	99.5	99.5	99.5	99.5	99.7	99.7	99.8	99.8	99.9	99.9	100.0				
500	99.8	99.5	99.7	99.7	99.8	99.4	99.3	99.2	99.6	99.5	99.7	99.6	99.3	99.4	99.5	99.5	99.5	99.3	99.7	99.7	99.8	99.8	99.9					
400	99.8	100.0	99.5	99.7	99.5	99.6	99.7	99.1	99.9	99.4	99.1	99.2	99.5	99.2	99.4	99.5	99.5	99.5	99.5	99.6	99.7	99.8						
300	99.8	100.0	100.1	99.6	99.5	98.5	99.5	99.5	99.7	99.2	99.0	99.8	99.3	99.0	99.2	99.3	99.6	99.5	99.5	99.4	99.6							
280	99.8	99.2	99.4	99.6	99.8	99.0	99.4	99.5	99.7	99.5	98.8	98.5	99.0	99.1	99.2	99.4	99.4	99.5	99.6	99.5								
260	99.8	99.5	99.7	99.3	99.1	99.0	99.5	99.2	99.8	99.4	98.7	98.5	99.0	99.0	99.2	99.4	99.3	99.5	99.5									
240	99.9	99.9	99.8	99.8	99.4	99.2	99.2	99.6	99.7	99.2	99.2	98.8	99.0	99.0	99.2	99.2	99.4	99.5										
220	100.0	99.2	99.6	99.5	99.5	99.3	99.2	99.5	98.9	99.4	99.3	98.4	98.6	99.0	99.1	99.2	99.0											
200	100.0	100.2	99.7	99.5	99.7	99.1	99.2	99.5	99.3	99.2	99.1	98.4	98.6	98.0	99.0	99.3												
180	100.1	100.3	99.9	99.8	99.9	98.7	99.9	99.4	99.5	99.0	99.2	98.0	98.7	98.7	99.0													
160	100.2	99.9	99.9	100.1	99.8	99.2	99.5	99.9	99.5	99.2	99.3	98.2	98.4	99.3														
140	100.0	100.1	99.5	99.9	100.0	99.5	99.9	100.0	99.4	99.5	99.0	98.2	98.1															
120	99.8	100.1	99.5	99.5	99.7	99.5	100.3	100.6	99.3	99.3	98.7	98.2																
100	99.5	99.5	99.3	99.5	99.5	99.8	100.5	101.5	100.0	99.3	98.6																	
90	99.8	99.3	99.1	99.5	99.4	99.5	101.2	101.2	99.7	99.2																		
80	102.1	99.0	98.9	99.1	100.1	100.5	101.4	101.6	99.9																			
70	105.2	98.2	99.1	98.7	100.6	101.0	102.0	102.1																				
60	106.7	99.4	97.6	98.5	101.0	101.6	102.1																					
50	105.0	103.5	100.1	99.3	102.0	101.8																						
40	102.4	103.3	104.8	99.6	100.4																							
30	106.2	101.7	102.3	102.3																								
20	96.8	102.2	102.9																									
10	96.3	97.7																										

FIG. 4.

The curves on the chart (Plate X) are remarkably smooth and regular for curves derived from a loading diagram with wheel concentrations. They are free from the kinks and irregularities found in charts based on the Cooper engine diagram, which are attributable to certain minor anomalies inherent in the Cooper loading.

If the proposed engine loading (M-60) is adopted, this chart, Plate X, can be incorporated in any set of printed specifications as a "time saver" in calculating live load stresses. It gives the maximum stresses produced by M-60, which are nearly equal to the maximum stresses producible by the heaviest existing locomotive loadings.

In Fig. 5, there is presented a greatly simplified form of chart of equivalent uniform loads devised by the writer. It is in the form of an isogonic or straight-line diagram constructed by a new empirical method.

As the relation for a given engine diagram between the equivalent uniform loads, q, and the span segments, l_1 and l_2, is not expressible as a continuous function, either logarithmic or otherwise, an ordinary nomographic diagram with straight axes cannot be applied. As an alternative, the writer decided to plot the values of q in a logarithmic field, and to use the mean curve thus obtained as the q-axis. The result is shown in Fig. 5. This simple isogonic chart gives the equivalent uniform loads for the proposed engine loading (M-60). It is, of necessity, slightly approximate; but the largest error resulting from its use is only ± 4.2 per cent. This diagram possesses advantages of simplicity and ease of reproduction, which counterbalance the advantage of greater precision of the preceding chart (Plate X).

An attempt to construct a similar nomographic diagram for the Cooper loading was abandoned, as the inherent peculiarities of that loading made it impossible to obtain an acceptable result.

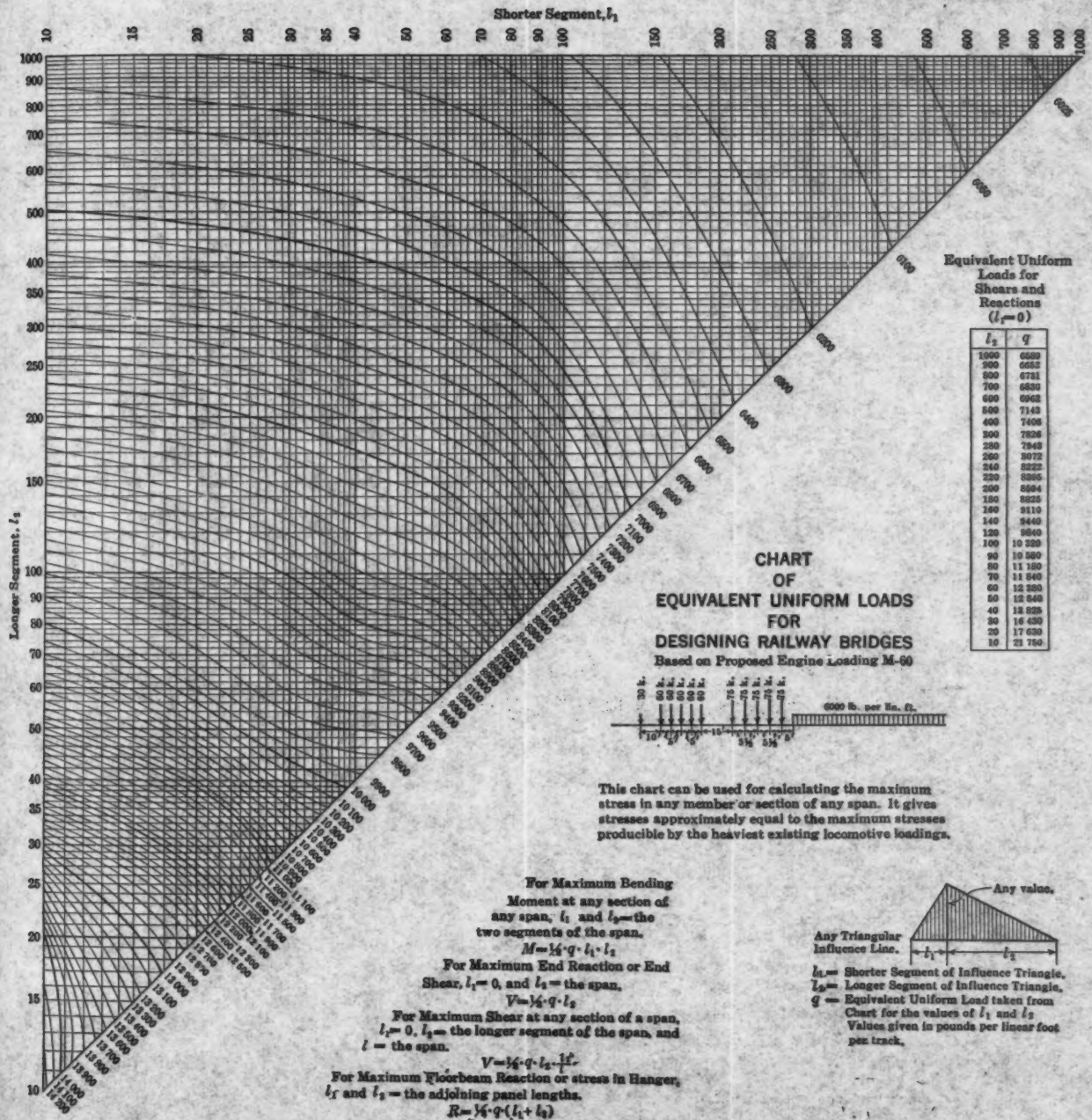
The small error of $\pm$ 4.2% in the isogonic chart (Fig. 5) could be still further reduced by substituting curved axes for l_1 and l_2. It requires merely a repetition of the operation by which the curved axis for q was obtained. By successive repetition of this procedure, a final isogonic chart may be obtained to give results with almost any desired degree of accuracy. The writer submits this as a suggested method for constructing an isogonic chart for any three variables between which there is a relation that is more or less regular, but which is not expressible by any continuous formula. This isogonic chart (Fig. 5) may be incorporated in specifications based on the proposed engine loading (M-60).

In Fig. 6 there is presented, in more familiar form, a graphic comparison between the different loadings under consideration, namely:

(1) The Cooper E-60 loading;
(2) The composite of the heaviest existing locomotives; and
(3) The proposed engine loading (M-60).

The stress-producing effects of these loadings are represented by graphs of equivalent uniform loads for end shears and center moments for spans of varying length.

PLATE X.
TRANS. AM. SOC. CIV. ENGRS.
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STEINMAN ON
LOCOMOTIVE LOADINGS
FOR RAILWAY BRIDGES.



Equivalent Uniform Loads for Shears and Reactions (l_1=0)

l_2	q
1000	6589
900	6655
800	6731
700	6830
600	6962
500	7143
400	7406
300	7826
280	7948
260	8072
240	8222
220	8395
200	8594
180	8825
160	9110
140	9440
120	9840
100	10 329
90	10 580
80	11 160
70	11 840
60	12 320
50	12 840
40	14 825
30	16 430
25	17 630
10	21 750

CHART OF EQUIVALENT UNIFORM LOADS FOR DESIGNING RAILWAY BRIDGES

Based on Proposed Engine Loading M-60

This chart can be used for calculating the maximum stress in any member or section of any span. It gives stresses approximately equal to the maximum stresses producible by the heaviest existing locomotive loadings.

For Maximum Bending Moment at any section of any span, l_1 and l_2 = the two segments of the span.

$$M = \tfrac{1}{2} \cdot q \cdot l_1 \cdot l_2$$

For Maximum End Reaction or End Shear, $l_1 = 0$, and l_2 = the span.

$$V = \tfrac{1}{2} \cdot q \cdot l_2$$

For Maximum Shear at any section of a span, $l_1 = 0$, l_2 = the longer segment of the span, and l = the span.

$$V = \tfrac{1}{2} \cdot q \cdot l_2 \cdot \tfrac{1}{l}$$

For Maximum Floorbeam Reaction or stress in Hanger, l_1 and l_2 = the adjoining panel lengths.

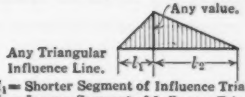
$$R = \tfrac{1}{2} \cdot q \cdot (l_1 + l_2)$$

l_1 = Shorter Segment of Influence Triangle.
l_2 = Longer Segment of Influence Triangle.
q = Equivalent Uniform Load taken from Chart for the values of l_1 and l_2 Values given in pounds per linear foot per track.

Equivalent Uniform Loads for Shears and Reactions
($l_1 = 0$)

l_2	1000	900	800	700	600	500	400	300	280	260	240	230	200	180	160	140	120	100	90	80	70	60	50	40	30	20	10
q	6589	6652	6731	6830	6962	7143	7406	7826	7943	8072	8222	8395	8594	8825	9110	9440	9840	10 320	10 580	11 180	11 840	12 380	12 840	13 825	16 430	17 630	21 750

This Chart can be used for figuring the maximum stress in any member or section of any span. It gives stresses approximately equal to the maximum stresses producible by the heaviest existing locomotive loadings.



l_1 = Shorter Segment of Influence Triangle.
l_2 = Longer Segment of Influence Triangle.
q = Equivalent Uniform Load taken from Chart for the values of l_1 and l_2
Values given in pounds per foot per track.

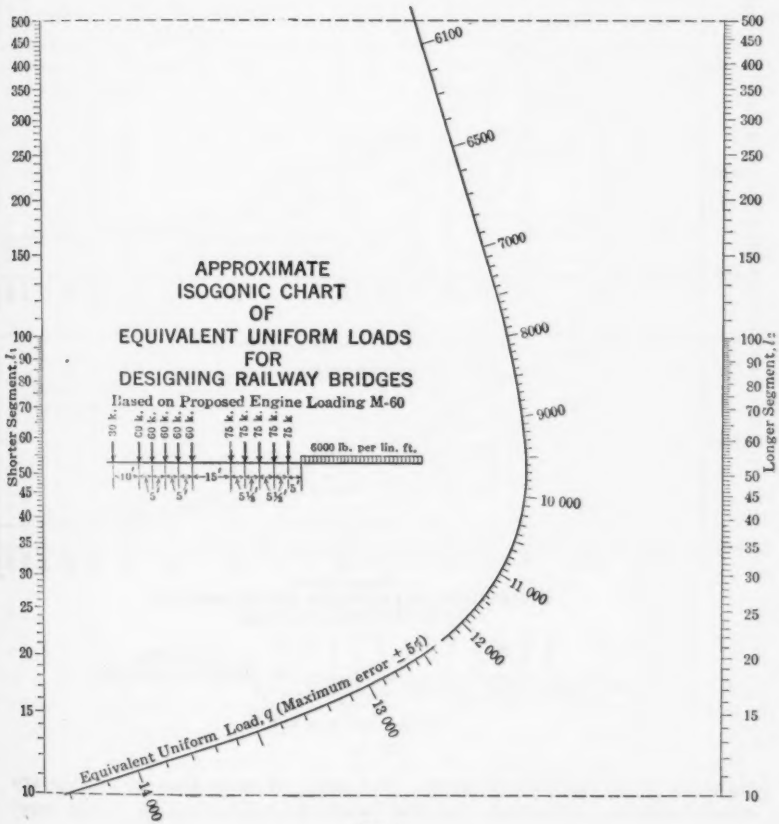


FIG. 5.

For both shears and moments, the Composite Standard is about 25% heavier than Cooper's E-60 at short spans and gradually approaches coincidence with the latter at very long spans. If Cooper's E-75 is substituted, it will cause an excess of 25% at long spans.

For end shears, there is the discrepancy of a few per cent., previously mentioned, between the Composite Standard and the proposed engine loading (M-60) at short spans. Except for spans of less than 24 ft., the

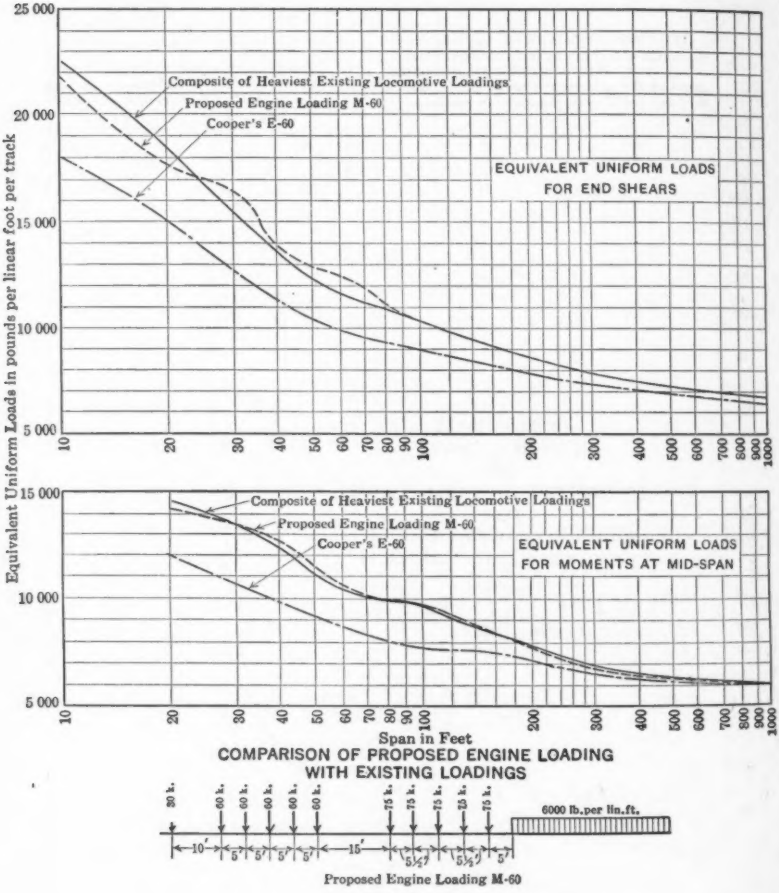


FIG. 6.

variation is on the side of safety. For spans of more than 90 ft., the two curves coincide, indicating that the proposed engine loading is the exact equivalent of the Composite Standard in this range.

In the graphs for center moments, there is a remarkable agreement between the proposed engine loading (M-60) and the Composite Standard. The

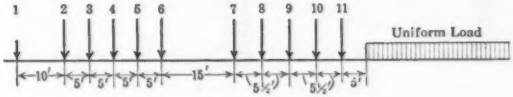
two curves follow each other throughout, with maximum variations of only 1 or 2%, except for one point having a difference of 2.9 per cent.

This agreement between the two curves is as close as could reasonably be demanded. Closer agreement cannot be secured without sacrificing the simplicity and uniformity of the proposed wheel-load diagram. The variation is so small that the proposed engine loading (M-60) may safely be accepted as a close equivalent of the composite of the heaviest existing locomotives.

The tabulation in Fig. 7 shows how the proposed engine loading can be diminished or augmented in fixed ratios to form different classes, in the same manner as the Cooper system of engines.

After conference with other engineers, the writer has decided to designate the proposed engine loading, previously derived, as equivalent to the heaviest modern locomotives, by the class symbol, M-60. The letter, M, stands for "Modern", "Motive Power", or "Mallet". The class number is made 60 to symbolize, in the conventional manner, the weights of the first group of driver axles and the weight of the uniform train load. An additional reason for the designation, 60, was to simplify the conversion to different classes.

PROPOSED ENGINE LOADING SYSTEM FOR DESIGN OF RAILWAY BRIDGES



Loads per Track

M-10 = 5	10	10	10	10	10	12.5	12.5	12.5	12.5	12.5	1000 lb. per lin. ft.
M-40 = 20	40	40	40	40	40	50	50	50	50	50	4000
M-50 = 25	50	50	50	50	50	62.5	62.5	62.5	62.5	62.5	5000
M-60 = 30	60	60	60	60	60	75	75	75	75	75	6000
M-70 = 35	70	70	70	70	70	87.5	87.5	87.5	87.5	87.5	7000
M-80 = 40	80	80	80	80	80	100	100	100	100	100	8000
M-90 = 45	90	90	90	90	90	112.5	112.5	112.5	112.5	112.5	9000

Loading M-60 will give stresses for all spans very close to the maximum stresses producible by the heaviest existing locomotive loadings. It is approximately equivalent to Cooper's E-75 for short spans and to Cooper's E-60 for long spans

FIG. 7.

Reducing the proposed engine loading to Class M-10, as a basic unit, there is a pilot axle of 5 k., five driver axles of 10 k., and five driver axles of 12.5 k., followed by a uniform load of 1 000 lb. per lin. ft. This simple loading is all that need be given in a specification. By multiplying this unit loading, M-10, by 4, 5, 6, 7, 8, or 9, the loadings, M-40, M-50, M-60, M-70, M-80, and M-90 are obtained.

Loading M-60 will give stresses for all spans very close to the maximum stresses producible by the heaviest existing locomotive loadings. To allow

for probable future increase in weight of traffic, the writer would recommend that Loading M-50 be specified in present-day specifications with a provision for an excess loading of 50%; or, what amounts to the same thing, Loading M-60 may be specified with a provision for an excess loading of 25 per cent.

TABLE 3.—EQUIVALENT COOPER RATINGS OF THE PROPOSED ENGINE LOADING (M-50).

Spans.	E-rating for center moments.	E-rating for end shears.
10	E-60	E-60
20	E-59	E-59
30	E-62	E-62
40	E-64	E-61
50	E-63	E-62
60	E-61	E-63
70	E-61	E-62
80	E-61	E-60
90	E-62	E-58
100	E-62	E-57
120	E-60	E-57
140	E-58	E-56
160	E-56	E-56
180	E-55	E-55
200	E-54	E-55
250	E-53	E-54
300	E-52	E-54
400	E-51	E-53
500	E-51	E-52
600	E-50	E-52
700	E-50	E-52
800	E-50	E-52
900	E-50	E-52
1 000	E-50	E-51

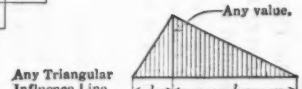
For spans up to 100 ft. the proposed Loading M-50 is about equivalent to Cooper's E-60. This is shown by Table 3 which gives the equivalent Cooper ratings of the proposed Loading M-50 for spans from 10 to 1 000 ft.

TABLE 4.—EQUIVALENT M-RATINGS OF THE COOPER E-60 LOADING.

Spans.	M-rating for center moments.	M-rating for end shears.
10	M-50	M-50
20	M-51	M-51
30	M-49	M-48
40	M-47	M-49
50	M-48	M-49
60	M-49	M-48
70	M-49	M-48
80	M-49	M-50
90	M-48	M-52
100	M-48	M-52
120	M-50	M-53
140	M-52	M-54
160	M-53	M-54
180	M-54	M-54
200	M-55	M-55
250	M-57	M-56
300	M-58	M-56
400	M-59	M-57
500	M-59	M-57
600	M-60	M-57
700	M-60	M-58
800	M-60	M-58
900	M-60	M-58
1 000	M-60	M-59

Shorter Segment l_1

Longer Segment l_2	0	10	20	30	40	50	60	70	80	90	100	120	140	160	180	200	220	240	260	280	300	400	500	600	700	800	900	1000
1000	6527	6522	6517	6512	6507	6502	6497	6493	6488	6484	6480	6471	6462	6455	6447	6400	6432	6425	6418	6412	6405	6376	6351	6330	6310	6294	6279	6265
900	6585	6577	6570	6565	6560	6553	6547	6542	6536	6531	6526	6515	6505	6496	6487	6478	6469	6461	6453	6445	6438	6405	6376	6351	6330	6310	6294	
800	6655	6646	6638	6631	6625	6616	6609	6602	6596	6590	6582	6570	6558	6545	6535	6525	6515	6505	6495	6485	6476	6436	6404	6375	6352	6329		
700	6745	6735	6725	6715	6705	6695	6685	6675	6667	6660	6653	6635	6620	6605	6593	6580	6567	6550	6543	6532	6521	6475	6436	6403	6375			
600	6865	6850	6835	6825	6815	6800	6785	6775	6765	6750	6740	6720	6700	6680	6665	6650	6635	6620	6605	6590	6576	6520	6474	6435				
500	7030	7010	6990	6970	6950	6935	6920	6905	6890	6875	6860	6830	6805	6780	6755	6735	6715	6695	6676	6660	6643	6575	6520					
400	7270	7240	7210	7180	7155	7130	7105	7080	7055	7035	7015	6975	6940	6905	6875	6845	6820	6795	6772	6750	6730	6640						
300	7650	7600	7550	7505	7460	7420	7380	7345	7310	7275	7240	7185	7130	7080	7040	7000	6965	6930	6900	6870	6840							
280	7760	7700	7650	7590	7540	7490	7450	7410	7370	7335	7300	7235	7175	7130	7080	7040	7000	6960	6925	6895								
260	7890	7820	7750	7690	7630	7580	7530	7480	7440	7400	7360	7290	7230	7175	7125	7080	7035	6995	6960									
240	8020	7940	7870	7800	7740	7680	7620	7570	7520	7470	7430	7350	7290	7225	7170	7120	7075	7030										
220	8190	8090	8000	7920	7850	7780	7720	7660	7600	7550	7500	7420	7350	7280	7220	7165	7115											
200	8380	8260	8160	8070	7980	7900	7830	7760	7700	7640	7585	7500	7420	7345	7280	7215												
180	8600	8460	8340	8230	8120	8040	7950	7870	7800	7730	7680	7580	7490	7410	7330													
160	8870	8700	8550	8420	8280	8190	8090	8000	7910	7850	7790	7670	7570	7480														
140	9190	8980	8800	8630	8480	8360	8240	8140	8050	7970	7900	7770	7650															
120	9600	9320	9090	8880	8700	8540	8400	8300	8200	8110	8030	7870																
100	10 110	9740	9420	9160	8940	8740	8610	8490	8370	8260	8160																	
90	10 400	9960	9600	9300	9040	8860	8660	8580	8450	8340																		
80	10 730	10 200	9780	9440	9150	8990	8830	8680	8540																			
70	11 070	10 440	9940	9550	9300	9110	8930	8760																				
60	11 400	10 630	10 050	9600	9420	9210	9000																					
50	12 340	11 280	10 530	9690	9480	9240																						
40	13 650	12 120	11 100	10 370	9820																							
30	15 600	13 200	11 760	10 800																								
20	18 600	14 400	12 360																									
10	24 000	15 000																										



l_1 = Shorter Segment of Influence Triangle.
l_2 = Longer Segment of Influence Triangle.
q = Equivalent Uniform Load taken from Chart for the values of l_1 and l_2. Values given in pounds per linear foot per track.

This table can be used for figuring the maximum stress in any member or section of any span. It gives stresses approximately equal to the maximum stresses producible by the heaviest existing locomotive loadings.

For Maximum Bending Moment at any section of any span, l_1 and l_2 = the two segments of the span.

$$M = \tfrac{1}{2} \cdot q \cdot l_1 \cdot l_2$$

For Maximum End Reaction or End Shear, $l_1 = 0$ and l_2 = the span.

$$V = \tfrac{1}{2} \cdot q \cdot l_2$$

For Maximum Shear at any section of a span, $l_1 = 0$. l_2 = the longer segment of the span and l = the span.

$$V = \tfrac{1}{2} \cdot q \cdot l_2 \cdot \frac{l_2}{l}$$

For Maximum Floorbeam Reaction or stress in Hanger, l_1 and l_2 = the adjoining panel lengths

$$R = \tfrac{1}{2} \cdot q \cdot (l_1 + l_2)$$

TABLE OF EQUIVALENT UNIFORM LOADS (PER TRACK) FOR DESIGNING RAILWAY BRIDGES

Based on Proposed Simplified Loading M-60

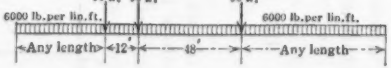


FIG. 8.

It will be observed, from Table 3, that the Cooper equivalent of M-50 diminishes from E-64 at short spans to E-50 at very long spans.

A reverse rating or conversion table is presented in Table 4 which gives the M-ratings of the Cooper E-60 loading for various spans.

According to Table 4, the M-equivalent of Cooper's E-60 ranges between M-47 at short spans and M-60 at long spans; but it is fairly well represented by M-50 for all spans below 100 ft.

Proposed Simplified Loading

As an alternative for the proposed engine diagram to represent the composite of the heaviest existing locomotives, the writer has devised an equivalent simplified loading which has certain advantages and may be preferred by some engineers.

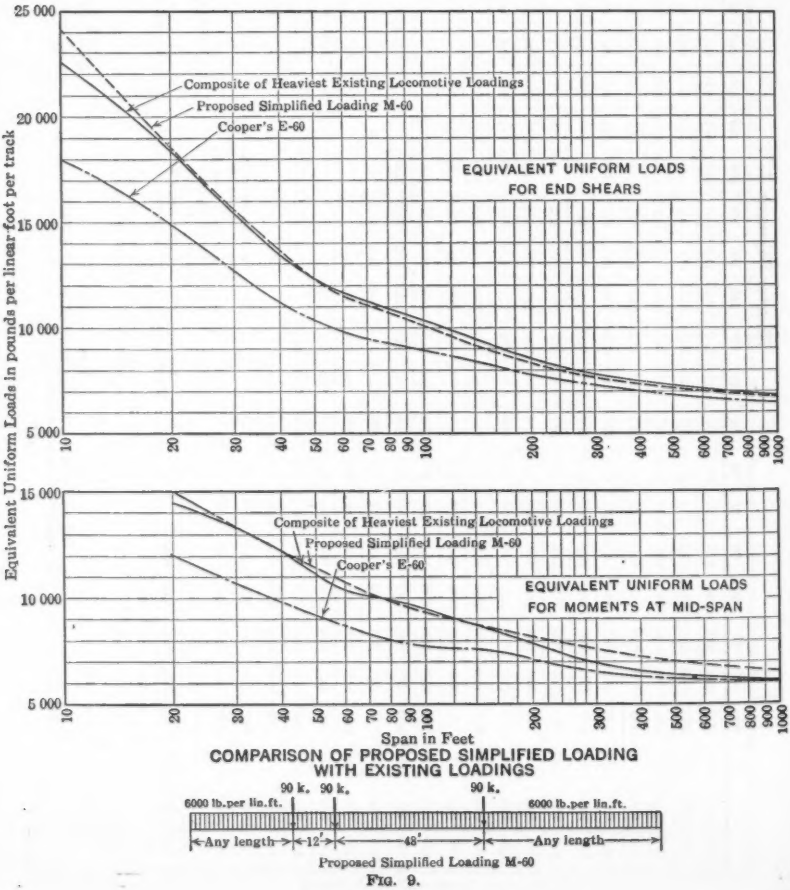


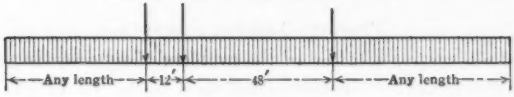
FIG. 9.

This proposed simplified loading (Figs. 8, 9, and 10) consists of a continuous uniform load of 6 000 lb. per lin. ft., on which is superimposed a floating group of excess concentrations. These concentrations, determined after a number of trials, consist of three 90 000-lb. loads spaced 12 ft. and 48 ft. apart, respectively.

The use of a simplified loading of this form greatly facilitates the computation of maximum stresses. Any stress is calculated by simply considering the uniform load and then adding the independent contribution of the group of concentrations. The latter contribution is quickly found; there are only three concentrations and they are easily placed and easily calculated. For stringer and floor-beam spans, only the first two concentrations enter.

To expedite still further the calculation of maximum stresses for this proposed simplified loading, and to permit direct comparison with the composite of the heaviest existing locomotives, a table of equivalent uniform loads is presented in Fig. 8. The computation of this table consumed only a small fraction of the time required for the similar table of equivalent uniform loads (Fig. 2) for the proposed engine loading, indicating the greater convenience and despatch of stress computations with the simplified loading.

PROPOSED SIMPLIFIED LOADING
UNIFORM LOAD WITH EXCESS CONCENTRATIONS
FOR DESIGN OF RAILWAY BRIDGES



Loads per Track

M-10	15	15	15	1000 lb. per lin. ft.
M-40	60	60	60	4000
M-50	75	75	75	5000
M-60	90	90	90	6000
M-70	105	105	105	7000
M-80	120	120	120	8000
M-90	135	135	135	9000

Loading M-60 will give stresses for all spans very close to the maximum stresses producible by the heaviest existing locomotive loadings. It is approximately equivalent to Cooper's E-75 for short spans and to Cooper's E-60 for long spans

FIG. 10.

Comparison of the values in this table (Fig. 8) with the equivalent uniform loads for the composite of the heaviest locomotives (Plate V) shows a sufficient agreement for all practical purposes. With the exception of some end shear values, where there is a maximum deficiency of 2.7%, the variations are on the side of safety. In other words, the proposed simplified loading will yield stresses equal to or somewhat greater than the heaviest existing locomotive loadings. Closer agreement could have been secured by varying the con-

centrated loads, but it was deemed preferable to keep the three concentrations equal for the sake of simplicity.

Moment and shear graphs comparing the simplified loading with existing loadings are presented in Fig. 9. The shear graph closely follows the curve for the Composite Standard, except for a variation of a few per cent. (on the safe side) at very short spans. The moment graph also follows the Composite Standard curve up to a span of about 200 ft., where it begins to deviate (on the safe side) with a maximum excess of about 10% at 500-ft. span.

It may be noted that the moment and shear graphs for the simplified loading are smoother than the corresponding curves for either the Composite Standard or the proposed engine loading.

To provide for different loading classes, similar to the different classes in the Cooper system, the proposed simplified loading can also be expressed in multiples of a smaller unit loading, as indicated in Fig. 10.

The simplified loading described, approximately equivalent to the heaviest modern locomotives, is called M-60. Accordingly, Loading M-10 consists of a uniform load of 1 000 lb. per lin. ft. with three excess concentrations of 15 k. each. Classes M-40, M-50, M-60, M-70, etc., are simply the corresponding multiples of Loading M-10.

For present-day specifications, as previously suggested, the writer would recommend Loading M-50 with provision for an increase of 50% or, what is equivalent, Loading M-60 with provision for an increase of 25 per cent.

Proposed Loading Formula

As a third alternative solution of the problem of representing the composite of the heaviest locomotives in more condensed form, the writer offers a loading formula which gives directly the equivalent uniform load at any section of any span.

In the opinion of other engineers besides the writer, this is an ideal solution of the problem, as a formula possesses many advantages over an engine diagram:

1.—It yields results of singular smoothness and regularity, unattainable with any diagram of wheel concentrations. The minor irregularities in stress variation due to unavoidable peculiarities in any concrete engine diagram are eliminated.

2.—It is a "time saver," as it yields maximum stresses directly without the labor of shifting a wheel-load diagram and calculating the contributions of individual concentrations.

3.—It facilitates the construction of tables, stress diagrams, and charts, and yields diagrams of greater simplicity, smoothness, and accuracy.

4.—It is more easily remembered than a wheel-load diagram, and saves repeated reference to the specifications.

5.—It makes the designer independent of reference books, as the ordinary slide-rule takes the place of all tables, diagrams, and charts.

6.—It is more flexible than a wheel-load diagram, being more easily modified to adjust it to any desired change in variation of equivalent loads with length of span.

7.—It is more scientific than an arbitrary loading diagram.

The formula devised by the writer to represent the maximum equivalent uniform loads of the heaviest existing engines, as tabulated on Plate V and plotted on Plate VI, is presented in Figs. 11, 12, 13, and 14, and Plates XI and XII, inclusive. It is as follows:

$$q = 5\,000 + 60\,000 \frac{\sqrt{l_2}}{l_1 + l_2} \quad \ldots\ldots(4)$$

in which,

l_1 = the shorter segment of the influence triangle for any stress;
l_2 = the longer segment of the influence triangle; and
q = the corresponding equivalent uniform load for maximum stress.

To obtain Equation (4), the writer platted the composite values of q for varying spans on logarithmic cross-section paper. He soon found that the graphs representing q — 5 000 were closely approximated by straight lines having a slope corresponding to the convenient exponent, $\frac{1}{2}$. The graph for end shears yielded Equation (5), as follows:

$$q = 5\,000 + \frac{60\,000}{\sqrt{\text{span}}} \quad \ldots\ldots(5)$$

and the graph for center moments yielded Equation (6):

$$q = 5\,000 + \frac{30\,000}{\sqrt{\frac{1}{2}\text{ span}}} \quad \ldots\ldots(6)$$

The resemblance between Equations (5) and (6) at once suggested that they could be considered as two special cases of one general formula, namely,

$$q = 5\,000 + \frac{r \times 60\,000}{\sqrt{r \times \text{span}}} \quad \ldots\ldots(7)$$

in which r = the position ratio in the span of the section considered. For end shears, the section is at the end of the span, consequently, $r = 1$, yielding Equation (5). For center moments, the section is at mid-span, consequently, $r = \frac{1}{2}$, yielding Equation (6). For maximum bending moment or chord stresses at the quarter-point of a span, $r = \frac{3}{4}$, and Equation (7) would yield:

$$q = 5\,000 + \frac{45\,000}{\sqrt{\frac{3}{4}\text{ span}}} \quad \ldots\ldots(8)$$

For maximum shear at the quarter-points of a span, the equivalent uniform load is the same as for end shear in a span only three-quarters as long, consequently,

$$q = 5\,000 + \frac{60\,000}{\sqrt{\frac{3}{4}\text{ span}}} \quad \ldots\ldots(9)$$

These examples show how simple the actual application of the formula becomes.

In order to facilitate the application of Equation (7) to finding maximum stresses of all kinds, it is expressed in more general form by introducing the symbols, l_1 and l_2, for the two segments of the loaded length of span. It then becomes:

$$q = 5\,000 + 60\,000 \frac{\sqrt{l_2}}{l_1 + l_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

which is the proposed loading formula.

Empirical formulas for equivalent uniform loads have been proposed in recent years, but they were deficient in one respect, namely, they expressed the equivalent uniform load as a function merely of the span without regard to the position of the member or section in the span.

The proposed formula, notwithstanding its simplicity, makes full provision for the necessary variation of equivalent uniform load for different sections or members in the same span. For instance, in the case of a 100-ft. span, the proposed loading formula yields values of q varying from 9 240 for bending moments or flange stresses at mid-span, to 11 000 for flange stresses near the ends of the span; and from 11 000 for end shears to 13 480 for shears near mid-span; a total range of variation of 46 per cent. This variation corresponds to the actual stress-producing effects of locomotive loadings, and the reproduction of this appropriate variation of equivalent uniform loads within a given span is an important advantage of the proposed loading formula.

The possibility of fairly close representation of the equivalent uniform loads for all sections of all spans by a single formula is a point of superiority of the proposed Composite Standard over the Cooper engine loading.

The equivalent uniform loads yielded directly by the proposed loading formula for various sections of various spans are tabulated on Fig. 11. This tabulation can be incorporated in printed specifications to expedite the calculation of stresses based on the formula.

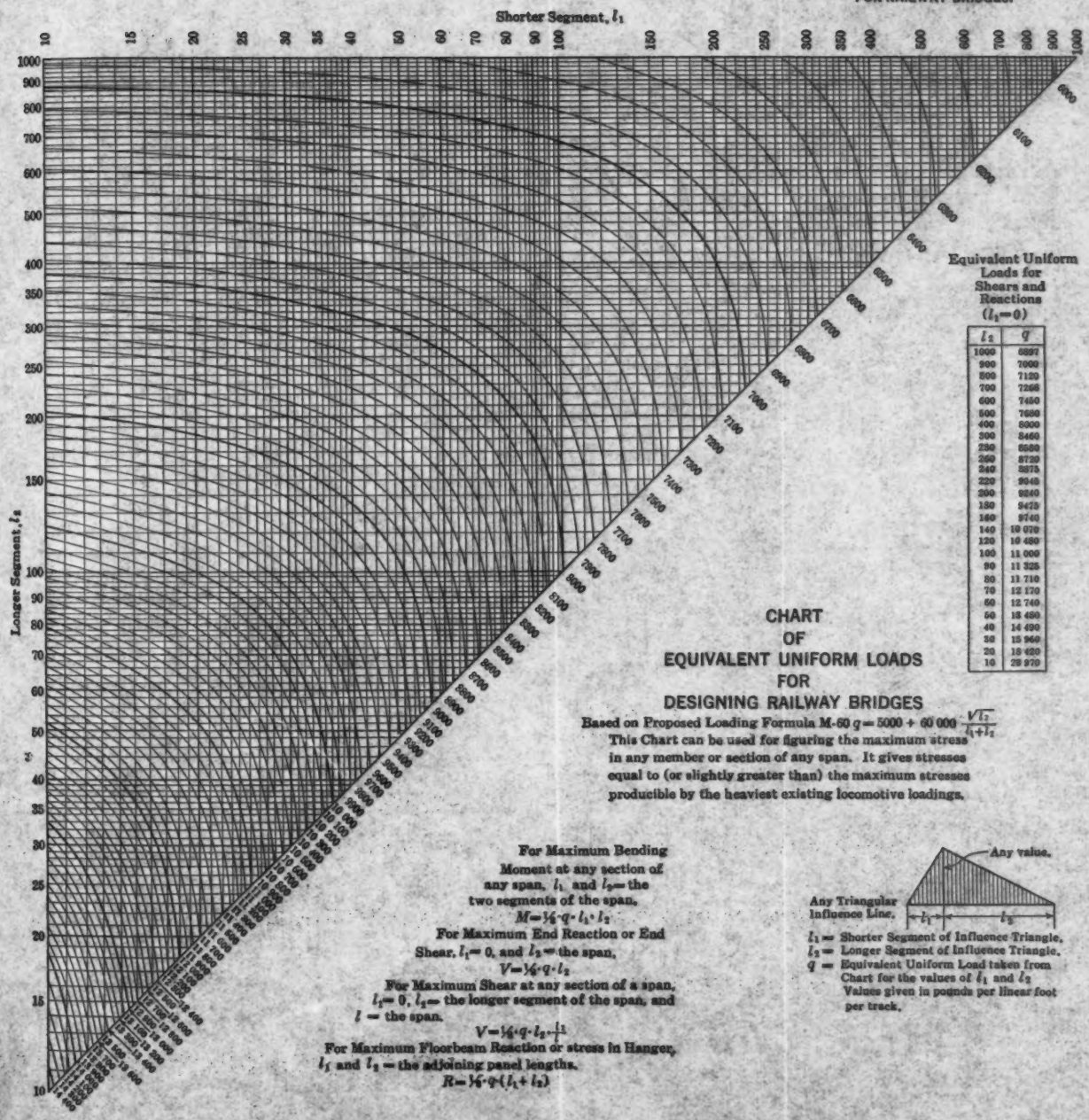
The tabulated values on Fig. 11 agree fairly well with those in the corresponding tables (Fig. 2 and Fig. 8) for the other proposed loadings. Except for only 4 or 5 values out of 405 in the table (Fig. 11), the values are on the safe side in comparison with the composite of the heaviest locomotives (Plate V). Closer agreement could have been secured by adopting a formula with an odd decimal exponent; but it appeared preferable to keep the simpler expression, although it varied somewhat (on the side of safety) from the exact values desired.

Platting the values tabulated on Fig. 11, one obtains for the proposed loading formula, a chart of equivalent uniform loads, Plate XI. This chart, although yielding nearly the same values throughout as the similar preceding charts, Plates VI and X, possesses a marked advantage in the perfect smoothness and systematic regularity of its curves. As irregularities in a chart may be regarded as weak spots in the stress-producing effects of the corresponding loading specification, the ideal regularity of the curves in Plate XI constitutes a strong argument in favor of the proposed loading formula.

General instructions for the use of the chart are given on Plate XI. More complete directions for finding maximum stresses of all kinds are given on the similar chart for the Cooper loading, Plate IV.

A much simpler chart for the proposed loading formula is an isogonic diagram presented in Plate XII. The equation between q, l_1, and l_2, is of such

PLATE XI.
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STEINMAN ON
LOCOMOTIVE LOADINGS
FOR RAILWAY BRIDGES.



Equivalent Uniform Loads for Shears and Reactions ($l_1 = 0$)

l_2	q
1000	6897
900	7000
800	7120
700	7268
600	7460
500	7690
400	8000
300	8460
280	8580
260	8720
240	8875
220	9040
200	9240
180	9470
160	9740
140	10 070
120	10 480
100	11 000
90	11 325
80	11 710
70	12 170
60	12 740
50	13 490
40	14 490
30	15 960
20	18 420
10	23 970

CHART OF EQUIVALENT UNIFORM LOADS FOR DESIGNING RAILWAY BRIDGES

Based on Proposed Loading Formula M-60 $q = 5000 + 60\,000 \frac{\sqrt{l_2}}{l_1 + l_2}$

This Chart can be used for figuring the maximum stress in any member or section of any span. It gives stresses equal to (or slightly greater than) the maximum stresses producible by the heaviest existing locomotive loadings.

For Maximum Bending Moment at any section of any span, l_1 and l_2 = the two segments of the span,

$$M = \tfrac{1}{8} \cdot q \cdot l_1 \cdot l_2$$

For Maximum End Reaction or End Shear, $l_1 = 0$, and l_2 = the span,

$$V = \tfrac{1}{2} \cdot q \cdot l_2$$

For Maximum Shear at any section of a span, $l_1 = 0$, l_2 = the longer segment of the span, and l = the span.

$$V = \tfrac{1}{2} \cdot q \cdot l_2 \cdot \frac{l_2}{l}$$

For Maximum Floorbeam Reaction or stress in Hanger, l_1 and l_2 = the adjoining panel lengths.

$$R = \tfrac{1}{2} \cdot q \cdot (l_1 + l_2)$$

l_1 = Shorter Segment of Influence Triangle.
l_2 = Longer Segment of Influence Triangle.
q = Equivalent Uniform Load taken from Chart for the values of l_1 and l_2
Values given in pounds per linear foot per track.

Shorter Segment, l_1

Longer Segment, l_2	0	10	20	30	40	50	60	70	80	90	100	120	140	160	180	200	220	240	260	280	300	400	500	600	700	800	900	1000
1000	6612	6550	6530	6480	6470	6450	6420	6410	6380	6360	6340	6278	6234	6202	6176	6156	6139	6126	6114	6104	6096	6067	6050	6039	6031	6026	6022	6019
900	6678	6600	6580	6560	6530	6500	6460	6450	6420	6400	6360	6305	6257	6220	6192	6170	6152	6137	6124	6113	6104	6072	6053	6042	6033	6028	6023	
800	6759	6680	6650	6620	6580	6540	6520	6500	6470	6450	6400	6338	6284	6244	6212	6187	6167	6150	6136	6124	6113	6078	6058	6045	6036	6029		
700	6863	6770	6740	6700	6660	6620	6600	6560	6530	6500	6450	6380	6318	6272	6236	6208	6185	6166	6150	6136	6125	6085	6062	6048	6038			
600	7000	6890	6850	6810	6760	6700	6690	6650	6610	6570	6530	6432	6361	6308	6266	6234	6207	6185	6167	6152	6139	6094	6068	6052				
500	7187	7060	7010	6950	6890	6820	6800	6760	6710	6670	6610	6502	6418	6355	6306	6267	6236	6210	6189	6171	6156	6104	6074					
400	7462	7280	7230	7150	7080	7000	6970	6920	6860	6810	6740	6598	6495	6418	6358	6311	6274	6244	6218	6196	6178	6117						
300	7898	7680	7570	7470	7380	7270	7230	7180	7100	7010	6920	6741	6607	6509	6433	6374	6327	6288	6257	6230	6208							
280	8019	7770	7660	7560	7448	7340	7300	7234	7153	7070	6970	6778	6636	6532	6452	6389	6340	6300	6266	6238								
260	8155	7890	7770	7660	7540	7430	7380	7320	7220	7120	7030	6819	6668	6557	6472	6406	6354	6312	6277									
240	8310	8020	7890	7762	7632	7510	7465	7390	7295	7190	7080	6865	6703	6585	6495	6425	6370	6323										
220	8489	8170	8020	7880	7750	7620	7560	7490	7370	7270	7150	6916	6742	6615	6519	6445	6387											
200	8696	8310	8170	8020	7872	7749	7680	7590	7477	7360	7230	6973	6785	6650	6546	6467												
180	8940	8550	8346	8187	8030	7890	7816	7710	7590	7454	7310	7038	6835	6688	6577													
160	9229	8780	8547	8380	8189	8050	7980	7860	7719	7570	7410	7111	6890	6731														
140	9574	9060	8792	8600	8387	8250	8162	8020	7870	7700	7530	7198	6953															
120	9990	9380	9080	8858	8617	8500	8389	8240	8063	7867	7670	7298																
100	10485	9780	9412	9160	8882	8790	8666	8490	8278	8070	7838																	
90	10926	9983	9590	9319	9030	8950	8828	8650	8390	8176																		
80	11625	10500	10012	9670	9375	9150	9009	8800	8543																			
70	12306	10956	10488	10064	9710	9450	9210	8985																				
60	12917	11429	10844	10417	10060	9720	9438																					
50	13500	11750	11143	10687	10250	9930																						
40	14906	12450	11312	10768	10359																							
30	16883	13625	12170	11083																								
20	18750	15000	13312																									
10	22500	15000																										



Any Triangular Influence Line

l_1 = Shorter Segment of Influence Triangle
l_2 = Longer Segment of Influence Triangle
q = Equivalent Uniform Load taken from Table for the values of l_1 and l_2 Values given in pounds per linear foot per track

This table can be used for calculating the maximum stress in any member or section of any span. It gives stresses approximately equal to the maximum stresses producible by the heaviest existing locomotive loadings

TABLE OF EQUIVALENT UNIFORM LOADS (PER TRACK) FOR DESIGNING RAILWAY BRIDGES

Proposed Engine Loading M-60



For Maximum Bending Moment at any section of any span, l_1 and l_2 = the two segments of the span

$$M = \tfrac{1}{8}\, q\, l_1\, l_2$$

For Maximum End Reaction or End Shear, $l_1 = 0$ and l_2 = the span

$$V = \tfrac{1}{2}\, q\, l_2$$

For Maximum Shear at any section of a span, $l_1 = 0$, l_2 = the longer segment of the span and l = the span

$$V = \tfrac{1}{2}\, q\, l_2 \cdot \frac{l_2}{l}$$

For Maximum Floorbeam Reaction or Stress in Hanger, l_1 and l_2 = the adjoining panel lengths

$$R = \tfrac{1}{2}\, q\, (l_1 + l_2)$$

FIG. 11.

form as to permit a perfect nomographic solution of conventional form. A straight-line chart of this form, yielding exact values for the equivalent uniform loads, cannot be constructed for any loading specification consisting of a diagram of axle concentrations. The isogonic diagram, Plate XII, is submitted merely as a suggestion. By varying the arrangement and scales of the three straight-line axes, other engineers can easily construct variations of this chart that may better suit their individual preference.

In the construction of the diagram, the values of span $(l_1 + l_2)$ and longer segment (l_2) are platted on ordinary logarithmic scales. The q-axis is a similar logarithmic scale, except that the constant, 5 000, has been added to each numbering.

One drawback of this isogonic chart, Plate XII, is the relatively small scale of the q-axis, which limits the precision attainable. The chart of curves, Plate XI, gives greater precision with equal, if not greater, expedition.

To overcome objections arising from the crowding of the graduations near one end of the q-axis in the exact nomographic diagram Plate XII, there is presented in Fig. 12 an isogonic chart of a more convenient form constructed for the same loading. This is an empirical and, therefore, slightly approximate, isogonic diagram of the type devised by the writer and previously described in connection with Fig. 5.

The smoothness and simplicity of this approximate nomographic chart (Fig. 12) for the proposed loading formula are unattainable with any other loading specification considered in this paper. The accuracy attainable is sufficient for most practical purposes, the greatest errors being less than ± 4 or 5 per cent. This chart can be easily reproduced and incorporated in any printed specifications based on the proposed loading formula. For the Cooper loading, a chart of this form is practically out of the question.

Moment and shear graphs are presented in Fig. 13 for the purpose of comparing the proposed loading formula with existing loadings. These graphs show that the proposed loading formula is a much closer and safer representation of the composite of the heaviest locomotives than the present Cooper loading.

For end shears, the graph for the proposed loading formula follows fairly close to the Composite Standard, with all variations on the side of safety.

For mid-span moments, the graph for the proposed loading formula is again a fairly close approximation to the Composite Standard, with a deficiency of only a few per cent. at short spans and a small variation on the safe side at long spans.

For stresses at intermediate sections of a span, the variations from the Composite Standard will be intermediate between the corresponding variations for end shears and center moments.

The moment and shear graphs for the proposed loading formula shown in Fig. 13 are obviously smoother curves than are attainable with any loading specification consisting of a diagram of wheel concentrations.

Fig. 14 shows how a system of loading classes similar to the Cooper system can be constructed from the proposed loading formula. Designating the

PLATE XII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXVI, No. 1520.
STEINMAN ON
LOCOMOTIVE LOADINGS
FOR RAILWAY BRIDGES.

Equivalent Uniform Loads for Shears and Reactions
($l_1 = 0$)

l_2	1000	900	800	700	600	500	400	300	250	240	220	200	180	160	140	120	100	90	80	70	60	50	40	30	20	10
q	6897	7000	7120	7260	7450	7690	8000	8460	8860	8720	8975	9045	9240	9475	9740	10 070	10 480	11 000	11 325	11 710	12 270	12 740	13 480	14 490	15 940	18 630	23 970



l_1 = Shorter Segment of Influence Triangle.
l_2 = Longer Segment of Influence Triangle.
q = Equivalent Uniform Load taken from Chart for the values of l_1 and l_2 Values given in pounds per lineal foot per track.

Total Length, $l_1 + l_2$

Equivalent Uniform Load, q

Longer Segment, l_2

ISOGONIC CHART
OF
EQUIVALENT UNIFORM LOADS
FOR
DESIGNING RAILWAY BRIDGES

Based on Proposed Loading Formula M-60-q = 5000 + 60 000 $\frac{\sqrt{l_2}}{l_1 + l_2}$

This Chart can be used for figuring the maximum stress in any member or section of any span. It gives stresses equal to (or slightly greater than) the maximum stresses producible by the heaviest existing locomotive loadings.

For Maximum Bending Moment of any section of any span, l_1 and l_2 = the two segments of the span.

$M = \frac{1}{2} \cdot q \cdot l_1 \cdot l_2$

For Maximum End Reaction or End Shear, $l_1 = 0$ and l_2 = the span

$V = \frac{1}{2} \cdot q \cdot l_2$

For Maximum Shear at any section of a span, $l_1 = 0$, l_2 = the longer segment of the span, and l = the span.

$V = \frac{1}{4} \cdot q \cdot l_2 \cdot \frac{l_2}{l}$

For Maximum Floorbeam Reaction or stress in Hanger, l_1 and l_2 = the adjoining panel lengths.

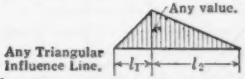
$R = \frac{1}{2} \cdot q \cdot (l_1 + l_2)$



Equivalent Uniform Loads for Shears and Reactions
($l_1 = 0$)

l_2	1000	900	800	700	600	500	400	300	280	260	240	220	200	180	160	140	120	100	90	80	70	60	50	40	30	20	10
q	6897	7000	7120	7200	7450	7660	8000	8460	8580	8720	8875	9045	9240	9475	9740	10 070	10 430	11 000	11 325	11 710	12 170	12 740	13 480	14 490	15 960	18 420	23 970

This Chart can be used for figuring the maximum stress in any member or section of any span. It gives stresses equal to (or slightly greater than) the maximum stresses producible by the heaviest existing locomotive loadings.



l_1 = Shorter Segment of Influence Triangle.
l_2 = Longer Segment of Influence Triangle.
q = Equivalent Uniform Load taken from Chart for the values of l_1 and l_2
Values given in pounds per foot per track.

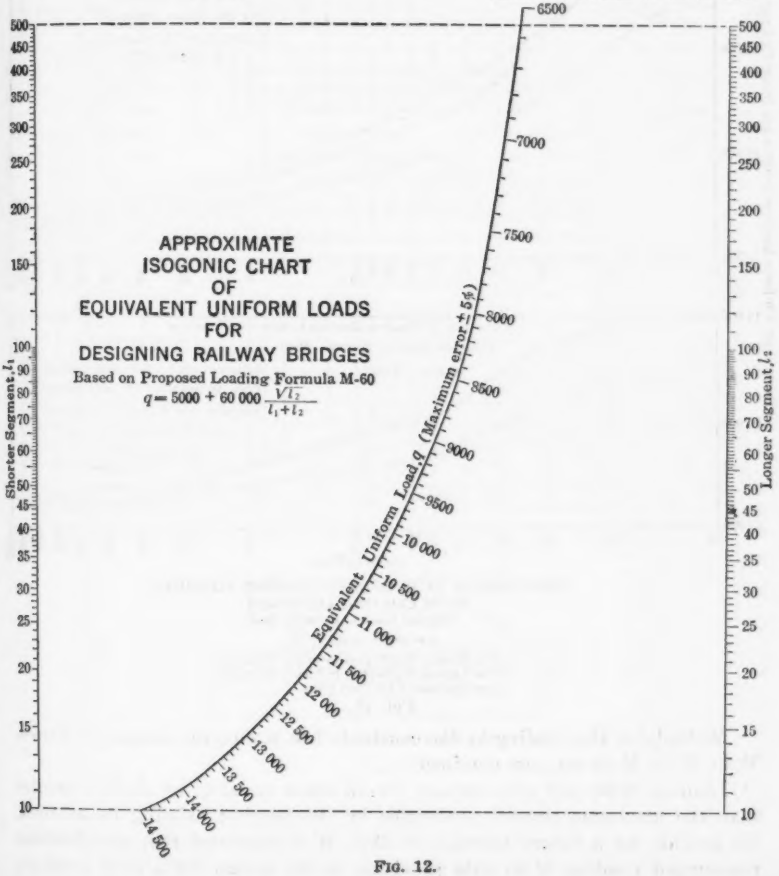
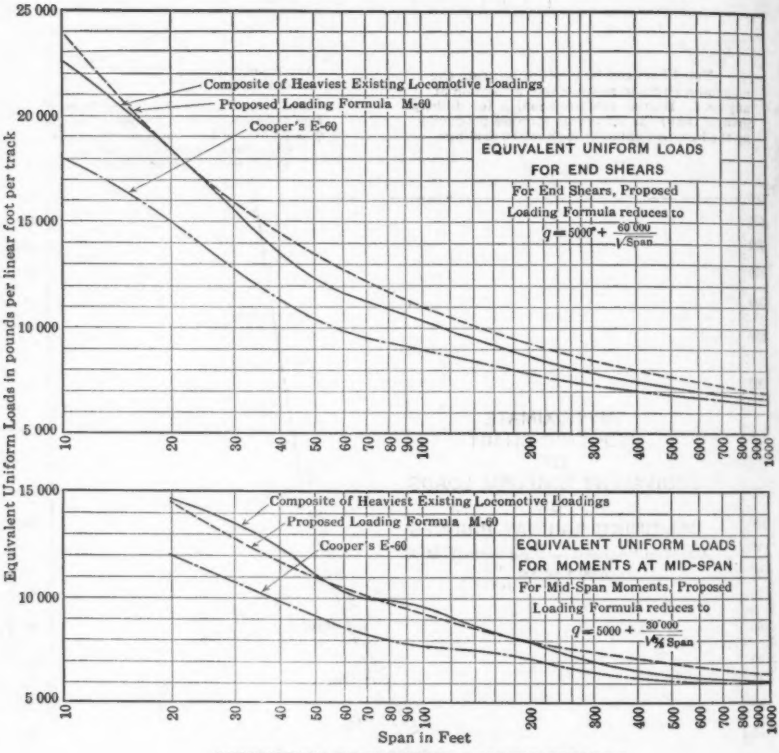


FIG. 12.

intensity of loading corresponding to the formula as derived from the Composite Standard as M-60, the unit base, or M-10 loading, becomes:

$$q = 833 + 10\,000\,\frac{\sqrt{l_2}}{l_1 + l_2} \qquad (11)$$



COMPARISON OF PROPOSED LOADING FORMULA WITH EXISTING LOADINGS

Proposed Loading Formula M-60

$q = 5000 + 60\,000\,\frac{\sqrt{l_2}}{l_1 + l_2}$

l_1 = Shorter Segment of Influence Triangle,
l_2 = Longer Segment of Influence Triangle,
q = Equivalent Uniform Load

FIG. 13.

Multiplying this loading by the constants 4, 5, 6, etc., the successive classes, M-40, M-50, M-60, etc., are obtained.

Loading M-60 will give stresses for all spans equal to or slightly greater than the maximum stresses producible by the heaviest existing locomotives. To provide for a future increase of 25%, it is suggested that specifications recommend Loading M-50 with provision in the design for a 50% overload, or Loading M-60 with a provision for a 25% overload.

Conclusion

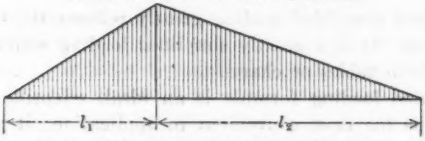
After showing the inadequacy of the Cooper system to represent modern loading conditions properly, the writer has constructed a Composite Standard covering the stress-producing effects of the heaviest existing locomotives and has been devised three alternative forms of specification conforming to this Composite Standard.

The advantages over the Cooper system of any loading specification based on the Composite Standard are:

1.—The proposed specification makes it possible to design bridges strong enough in all their parts, without waste, for the heaviest existing locomotive loadings, without requiring separate computations for each loading.

2.—The proposed specification gives bridges of uniform strength in all

PROPOSED LOADING FORMULA FOR EQUIVALENT UNIFORM LOADS FOR DESIGN OF RAILWAY BRIDGES



l_1 = Shorter segment of Influence Triangle
l_2 = Longer segment of Influence Triangle
q = Equivalent Uniform Load for the values of l_1 and l_2

M-10	$q = 833 + 10\,000$	$\frac{\sqrt{l_1 l_2}}{l_1 + l_2}$
M-40	$q = 3333 + 40\,000$	$\frac{\sqrt{l_1 l_2}}{l_1 + l_2}$
M-50	$q = 4166 + 50\,000$	$\frac{\sqrt{l_1 l_2}}{l_1 + l_2}$
M-60	$q = 5000 + 60\,000$	$\frac{\sqrt{l_1 l_2}}{l_1 + l_2}$
M-70	$q = 5833 + 70\,000$	$\frac{\sqrt{l_1 l_2}}{l_1 + l_2}$
M-80	$q = 6666 + 80\,000$	$\frac{\sqrt{l_1 l_2}}{l_1 + l_2}$
M-90	$q = 7500 + 90\,000$	$\frac{\sqrt{l_1 l_2}}{l_1 + l_2}$

Loading M-60 will give stresses for all spans very close to the maximum stresses producible by the heaviest existing locomotive loadings. It is approximately equivalent to Cooper's E-75 for short spans and to Cooper's E-60 for long spans

Fig. 14.

their parts for present and prospective future loadings, without requiring different load ratings for different members in a span.

3.—The proposed specification gives designs of uniform strength for bridges of all spans under present and prospective future loadings, without requiring different load ratings for different lengths of span.

4.—The proposed specification, being based on the heaviest modern locomotives and train loads, better represents the future development to be expected in loadings than a specification based on the engines and train loads of thirty years ago.

In addition, the proposed specification, in the three forms presented by the writer, offers advantages over the Cooper loading in simplicity, regularity, ease of computation, and greater facility of diagrammatic representation.

Of the three alternative forms developed in this paper for the proposed new specification, each has distinctive features to recommend it:

1.—The proposed loading diagram represents the smallest departure from present methods, involving merely the substitution of a new and simpler wheel-load diagram for the one now in use. It will best meet the requirements of engineers who prefer a concrete picture of a locomotive as a loading specification. Furthermore, it affords the closest agreement throughout the entire range of spans with the composite of the heaviest existing locomotives.

2.—The proposed simplified loading greatly reduces the time and labor of calculating stresses. It is a greatly simplified loading which can be handled expeditiously without tables or charts.

3.—The proposed loading formula is an ideal, scientific form of loading specification. It is the most convenient in application; it yields tables, diagrams, and charts of unequalled simplicity and regularity; or, if tables and charts are not at hand, any maximum stress can be found by a simple operation on the ordinary slide-rule. It is a "short-cut" specification, virtually prescribing stresses rather than loads.

If this paper only serves to direct attention to the inadequacy of existing loading specifications, and to indicate the general lines of attack in determining a new specification to be adopted, the writer will feel repaid for the labor expended.

DISCUSSION

C. D. PURDON,* M. AM. SOC. C. E. (by letter).—The author appears to assume that, at some time, all railroads will have to provide for such engines as the enormous ones described in the paper. Although it is true that cars of any kind may be encountered on any railroad, they may be pulled by small engines, and bridges should provide for these as a minimum. Such engines as those described in the paper are for exceptional places, very heavy tonnage, and heavy grades, and are not likely to be found on roads which do not have such grades and business. The load of heavy cars, expressed as weight per foot, does not express their effect on bridges. For instance, the car of the Norfolk and Western Railroad Company, of 5 346 lb. per ft., has a maximum effect in shear of a little more than E-48 and in moment, of E-47. A gondola of the Pennsylvania System, of 3 980 lb. per ft., overloaded 10%, has a maximum effect of E-44.5 when in solid trains, but if alternated with 80 000-lb. cars fully loaded, it has an effect of E-31 on spans of 60 ft. and more.

The typical engine described by the author does not resemble an engine and is only a load per foot and his M-loading might as well be used. This would require a purchaser to turn the engine he desires into M-loading. Although the tractive power of M is not given—as it generally runs about 22% of weight on drivers—an M-loading, on a line with maximum grades of 0.5%, would haul more cars than could be handled.

In preparing the 1920 specifications, the Iron and Steel Structures Committee of the American Railway Engineering Association canvassed carefully all kinds of loading and finally decided on the Cooper series as being the most suitable. The specification is for spans of 300 ft. and less; for longer spans, the same loading "per foot" could be used; and, for long spans, the floor and some members are designed by the Cooper loading, but the trusses are on a "per foot" basis.

The author states that his M-loading provides for future increase. The A.R.E.A. loading also allows for a future increase by using a unit stress of 16 000 lb., after adding impact. This has been criticized by some engineers. Why not use the actual safe stress of, say, 20 000 or 22 000 lb. per sq. in.? If the M-loading allows for increase, consider it with a unit stress of 20 000 lb., which is the same as adding 25% to the Cooper loading. Then, taking Fig. 6 and adding 25% to the Cooper E-60 brings a curve practically the same as M-60 up to 80 ft., after which it is a little above that loading. Few railroad companies in the West would care to build their trestle bridges for such a loading.

Some years ago, the writer made a comparison of engines of different kinds, with the Cooper loading, using E-55 as a base. Through the courtesy of the Baldwin Locomotive Works, many diagrams were obtained. These diagrams were reduced to an equivalent load of 55 000 lb. on each driving axle, and either the shear or the moment was plotted, the highest value being used, but as the following load is only an assumption, the supposed bridge was a deck

* Cons. Engr., St. L. S. W. Ry., St. Louis, Mo.

girder up to 120 ft. This diagram* indicates that the engines of the Santa Fé type would make a fairly regular curve up to 120 ft., as they probably would not be "double-headed". A following load of about 7 000 lb. (which is about the weight per foot of the engine itself) might be used. With regard to the Cooper loading not representing the actual, consider the effect of a Consolidation engine (double-headed), rated at about E-42, on a 200-ft. span originally built for the Cooper A, which is approximately E-25. The Cooper rating will range from E-37.58 to E-39.59 for the lower chord members, and from E-37.65 to E-45.61 for the web members.

The Santa Fé type seems to be a favorite; Record 88 of the Baldwin Locomotive Works, which is dated 1917, states that between 1903 and 1906, the Atchison, Topeka and Santa Fé Railway Company had bought 160 locomotives and had since bought 32 more; that the Chicago, St. Paul, Minneapolis and Omaha Railway Company had bought 2 engines; the Texas and Pacific Railway Company, 26; the Duluth, Missabe and Northern Railroad, 6; the Chicago Great Western, 7; the Union Pacific, 27; the Chicago, Burlington and Quincy, 66; the Lehigh Valley, 76; the Southern, 55; the St. Louis, San Francisco, 60; the Bessemer and Lake Erie, 20; the Baltimore and Ohio, 30; and the Erie, 20; and, in June, 1922, 50 locomotives were on their way to the Southern Pacific.

The average weight of these locomotives (all super-heater) is 548 745 lb.; on drivers, 294 482 lb.; tractive power, 72 973 lb. (equal to 24.78%); length, 81 ft. 9 in.; weight per foot, 6 596 lb.; and weight per pound of tractive power, 7.52. Comparing these with nine Consolidated locomotives, taken at random, we have: Weight, 336 155 lb.; on drivers, 191 711 lb.; tractive power, 46 008 lb. (equal to 24%); length, 68 ft.; weight per foot, 4 938 lb.; and weight per pound of tractive power, 7.36. Six ten-wheelers gave 22% tractive power, with an average weight per pound of tractive power of 10.22. This indicates that the Santa Fé type carries no more load in proportion than a Consolidated engine, not superheated, and less than a ten-wheeler.

ROBERT C. STRACHAN,† M. AM. SOC. C. E. (by letter).—The author has performed a valuable work by his analysis of the bases underlying much present practice in specifying train loads, and by making evident some of the inconsistencies involved in attempting to adapt to present-day conditions an ancient generalization, which, although excellent in its day, has outlived its usefulness. His constructive suggestions merit the careful consideration of the bridge engineer.

A reading of the paper compels one to realize that, although stresses and sections are computed to the "twentieth part of one poor scruple", rivets numbered and placed with the minutest attention to primary, secondary, or even tertiary, stress, and impact coefficients determined with the utmost refinement, the completed structure may be far from good, if not designed for the loads actually to be carried at once, or reasonably to be expected during its life.

The author's recommendations as to a new specification of the amount and distribution of train loads, based on his study of many pertinent data, are

* Published in *Engineering News*, of October 23, 1913, but without the Erie Triplex.
† New York City.

advanced in three forms; the second and third of these forms are derived from the first, and are shown to possess advantages due to possible simplifications in mathematical processes. These advantages are of undoubted weight; but the second and third forms are, as it were, "implicit functions" of the first, reflecting all its possible imperfections, and tending to remove the nub of the argument a step farther in any discussion of the merits of the author's suggestions.

In drawing any specification, it would seem preferable to state the fundamental requirements, rather than the results of conclusions drawn therefrom by methods which themselves may be open to argument. Accordingly, confining the discussion to the proposed conventional axle-load diagram (Fig. 7), the author's determination of M-60 after careful study of the stress-producing effects of the heaviest existing locomotives, and his subsequent building up of a series of M-diagrams bearing simple ratios to M-60, in the manner of the Cooper loading, is an important step in the direction of introducing a scientific and modern standard load system for the economical design of railroad bridges and for the rating or comparison of their carrying capacities; a system of which the need is admittedly great in consequence of the changes in locomotive construction which have taken place since the Cooper system of loading was devised.

Although professional opinions may differ as to the degree of approximation to the truth attained in the use of the proposed conventional system, its adoption would tend toward augmented economy and toward the elimination of many unprofitable complications inseparable from the use of an obsolete load specification.

Although it is the writer's opinion that neither the second nor the third form of the author's recommendations should appear as a load specification, both of them, with the accompanying diagrams, will commend themselves as "short-cuts" for use under suitable circumstances.

VICTOR H. COCHRANE,* M. AM. SOC. C. E. (by letter).—The author's tables and diagrams clearly show that, in the Cooper series, there is a lack of balance between the locomotive axle concentrations and the uniform train load. The maximum axle load is ten times as great as the train load per linear foot, whereas in the case of the heaviest loadings now in use this ratio is about 13. For some locomotive loadings the discrepancy is not so great; for instance, the last three loadings of the group of seven presented by the author are fairly well represented by the Cooper E-60 loading. However, if these same locomotives were followed by a lighter uniform loading, such as the 210 000-lb. cars assumed by a committee of the American Railway Engineering Association† in its study of the moments and shears produced by modern locomotives and cars, the Cooper loading would again be out of balance. It seems desirable, therefore, to make the axle loads heavier in proportion to the uniform train load than is done in the Cooper series.

* Cons. Engr. (Cochrane, Willrich and Johnson), Tulsa, Okla.
† *Proceedings*, Am. Ry. Eng. Assoc., Vol. 21.

Modern locomotives differ so greatly in the arrangement and spacing of axle loads that no single type can be found to represent all of them, unless it is a more or less conventional system, such as the author's first alternative loading. A composite loading derived from a number of different types of heavy locomotives will undoubtedly be more nearly representative of future loadings than any single loading series, and, consequently, will make more adequate provisions for future developments in locomotive design.

Of the three alternative loadings proposed, the writer prefers the loading formula, as it is by far the simplest and most direct. There seem to be two principal objections to its use: First, that it does not give a graphic picture or diagram of the axle concentrations and uniform load to which it corresponds; and, second, that it is not in close enough agreement with the composite loading. These objections are outweighed by the advantages which are well stated by the author.

In regard to the first objection, which relates to the lack of a loading diagram, there is no reason why an approximate equivalent loading, such as the author's first alternative, should not be used in connection with the formula to show approximately the loads provided for.

Concerning the second objection, that relating to the differences between the formula loading and the composite loading, it may be pointed out that, inasmuch as different locomotives in use give widely different results, and as there is no way of knowing exactly the future trend of locomotive design, these discrepancies are of minor consequence.

However, if desirable to do so, it is practicable to devise a formula which will agree much more nearly with the composite loading than that of the author. Such a formula might not be quite as simple in appearance as that of the author, but it would be practically as convenient to use. The following is suggested as an alternative to the author's formula, using the same notation:

$$q = 4\,400 + \frac{50\,000}{\sqrt{l_1 + l_2}} - 600 - \frac{l_2 - l_1}{l_2 + 3\,l_1}\left(\begin{matrix}\text{plus } 3\,000 - 60\,l_2, \text{ for} \\ \text{shears when } l_2 < 50\end{matrix}\right).$$

This formula reduces to:

$$q = 5\,000 + \frac{50\,000}{\sqrt{l_2}} \text{ (plus } 3\,000 - 60\,l_2, \text{ for } l_2 < 50), \text{ for shears;}$$

$$q = 4\,600 + \frac{50\,000}{\sqrt{L}}, \text{ for quarter-span moments;}$$

$$q = 4\,400 + \frac{50\,000}{\sqrt{L}}, \text{ for mid-span moments.}$$

This formula agrees with the composite loading practically as closely as the first alternative loading. Table 5 shows the values obtained by the author's formula and by that of the writer, also the percentages of error based on the maximum values of the composite loading. It will be noted that for moments this formula gives a series of curves parallel to the end shear curve. The Cooper loading is different in this respect, as will be seen.

TABLE 5.—VALUES OF EQUIVALENT LOADS AS DERIVED FROM HEAVIEST LOCOMOTIVE LOADINGS AND AS GIVEN BY PROPOSED FORMULAS.

Span length, in feet.	End Shears.					Moments at Quarter Span.					Moments at Mid-Span.				
	Heaviest locomotive loading, in pounds. (Plate VI).	As given by author's formula.		As given by proposed formula.		Heaviest locomotive loading, in pounds. (Plates V and VI).	As given by author's formula.		As given by proposed formula.		Heaviest locomotive loading, in pounds. (Plate VI).	As given by author's formula.		As given by proposed formula.	
		Amount.	Percentage of variation.	Amount.	Percentage of variation.		Amount.	Percentage of variation.	Amount.	Percentage of variation.		Amount.	Percentage of variation.	Amount.	Percentage of variation.
10.....	22 600	23 970	+ 6.1	23 210	+2.7	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
20.....	18 400	18 420	+ 0.1	17 980	−2.3	14 670	16 620	+13.3	15 780	+7.6	14 580	14 490	−0.6	15 580	+6.9
30.....	15 460	15 960	+ 3.2	15 330	−0.8	13 900	14 490	+ 4.3	13 730	−1.2	13 480	12 750	−5.4	13 530	+0.4
40.....	13 500	14 490	+ 7.3	13 500	0.0	12 700	13 220	+ 4.1	12 510	−1.5	12 320	11 710	−4.9	12 310	−0.1
50.....	12 230	13 480	+10.2	12 070	−1.3	11 420	12 350	+ 8.1	11 670	+2.2	11 400	11 000	−3.8	11 470	+0.6
60.....	11 600	12 740	+ 9.8	11 450	−1.3	10 750	11 710	+ 8.9	11 050	+2.8	10 320	10 480	+1.5	10 850	+5.1
70.....	11 260	12 170	+ 8.1	10 980	−2.5	10 360	11 210	+ 8.2	10 580	+2.1	10 120	10 070	−0.5	10 380	+2.6
80.....	10 940	11 710	+ 7.0	10 590	−3.2	10 100	10 800	+ 6.9	10 190	+0.9	9 900	9 740	−1.6	9 990	+0.9
90.....	10 600	11 325	+ 6.8	10 270	−3.1	9 890	10 480	+ 6.0	9 870	−0.2	9 720	9 470	−2.6	9 670	−0.5
100.....	10 374	11 000	+ 6.0	10 000	−3.6	9 670	10 200	+ 5.5	9 600	−0.7	9 480	9 240	−2.5	9 400	−0.8
120.....	9 860	10 480	+ 6.3	9 560	−3.0	9 200	9 740	+ 5.9	9 160	−0.4	9 040	8 870	−0.8	8 960	−0.9
140.....	9 440	10 070	+ 6.7	9 230	−2.2	8 860	9 390	+ 6.0	8 880	−0.3	8 640	8 590	−0.6	8 630	−0.1
160.....	9 090	9 740	+ 7.1	8 950	−1.5	8 500	9 110	+ 7.2	8 550	+0.6	8 400	8 350	−0.6	8 350	−0.6
180.....	8 815	9 475	+ 7.5	8 730	−1.0	8 300	8 870	+ 6.9	8 330	+0.4	8 120	8 160	+0.5	8 130	+0.1
200.....	8 590	9 240	+ 7.6	8 540	−0.6	8 090	8 670	+ 7.2	8 140	+0.6	7 840	8 000	+2.0	7 940	+1.3
220.....	8 397	9 045	+ 7.7	8 370	−0.3	7 920	8 500	+ 7.3	7 970	+0.6	7 600	7 860	+3.4	7 770	+2.2
240.....	8 228	8 875	+ 7.9	8 230	0.0	7 720	8 350	+ 8.2	7 830	+1.4	7 360	7 740	+5.2	7 630	+3.7
260.....	8 086	8 720	+ 7.8	8 100	+0.2	7 640	8 220	+ 7.6	7 700	+0.8	7 200	7 630	+6.0	7 500	+4.2
280.....	7 958	8 580	+ 7.8	7 990	+0.4	7 510	8 110	+ 8.0	7 590	+1.1	7 040	7 540	+7.1	7 390	+5.0
300.....	7 842	8 460	+ 7.9	7 890	+0.6	7 380	8 000	+ 8.4	7 490	+1.5	6 900	7 450	+8.0	7 290	+5.6
400.....	7 423	8 000	+ 7.8	7 500	+1.0	6 900	7 600	+10.1	7 100	+2.9	6 496	7 120	+9.6	6 900	+6.2
500.....	7 160	7 680	+ 7.3	7 240	+1.1	6 640	7 320	+10.2	6 840	+3.0	6 330	6 900	+9.0	6 640	+4.9
600.....	6 981	7 450	+ 6.7	7 040	+0.8	6 450	7 120	+10.4	6 640	+2.9	6 220	6 730	+8.2	6 440	+3.5
700.....	6 846	7 268	+ 6.2	6 890	+0.6	6 330	6 960	+10.0	6 490	+2.5	6 180	6 600	+6.8	6 290	+1.8
800.....	6 744	7 120	+ 5.6	6 770	+0.4	6 248	6 840	+ 9.5	6 370	+2.0	6 124	6 500	+6.2	6 170	+0.7
900.....	6 665	7 000	+ 5.0	6 670	+0.1	6 200	6 730	+ 8.5	6 270	+1.1	6 100	6 410	+5.1	6 070	−0.5
1 000.....	6 600	6 897	+ 4.5	6 580	−0.3	6 170	6 640	+ 7.6	6 180	+0.2	6 080	6 340	+4.3	5 980	−1.6

The Cooper E-60 loading can be represented by a similar formula with practically the same degree of accuracy that is shown in Table 5 for the M-60 loading. The general formula is:

$$q = 5\,000 + \frac{30\,000 + 10\,000\,\dfrac{l_2 - l_1}{l_2 + 3\,l_1}}{\sqrt{l_1 + l_2}},$$

which reduces to:

$$q = 5\,000 + \frac{40\,000}{\sqrt{l_2}}, \text{ for shears;}$$

$$q = 5\,000 + \frac{33\,300}{\sqrt{L}}, \text{ for quarter-span moments;}$$

$$q = 5\,000 + \frac{30\,000}{\sqrt{L}}, \text{ for mid-span moments.}$$

The degree of accuracy of this formula may be determined by comparison with Plate IV.

The author's formula becomes:

$$q = 5\,000 + \frac{60\,000}{\sqrt{l_2}}, \text{ for shears;}$$

$$q = 5\,000 + \frac{52\,000}{\sqrt{L}}, \text{ for quarter-span moments;}$$

$$q = 5\,000 + \frac{42\,400}{\sqrt{L}}, \text{ for mid-span moments.}$$

It seems likely that any system of loading that may be adopted, can be represented closely enough for practical purposes by a general formula similar to that just mentioned.

The author's suggestion in regard to the use of a formula (or charts and tables corresponding thereto), is a valuable one, and is worthy of adoption.

ALBERT LUCIUS,* M. AM. SOC. C. E. (by letter).—The writer believes the tendency in future bridge loadings will not be toward increased driver axle loads, as the economical limit of rail service is against it, but that the number of drivers will be increased where more traction unit is demanded. He also believes that when the electrification of railroads takes place, as sooner or later it will, new motor diagrams will be developed, aiming at a more uniform and equal maximum of motor axle and car axle loads, with a view to reducing maintenance-of-way costs of roadbed and structures.

Referring to the author's request for an expression of preference concerning several alternate propositions for railway bridge loadings in specifications, the writer's preference is as follows: Specify the maximum local axle loads and spacing, the heaviest typical locomotive diagram, and the train load per foot of track.

This expression of preference is without prejudice to the value and correctness of the author's carefully prepared paper and his conclusions and recom-

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mendations as contained therein. The writer begs to express his full appreciation of the labor involved and of the service which the author has rendered to the Profession by its publication.

B. A. WORTHINGTON,* ESQ. (by letter).—Relative to the author's suggestion of a wheel loading for locomotives to be used in bridge design to displace the Cooper loading, there is no doubt but that the Cooper loading diagram was scientifically developed, and that at the time of its adoption it was the best and most efficient specification for the locomotives in use. However, the present type of heavy locomotive has outgrown the Cooper loading diagram and the writer thinks the point made by Mr. Steinman is well taken, namely, that the Cooper loading diagram will have to be modified materially if the stresses in railway bridges are to be uniform and no waste in material is to occur.

The importance of making this change is brought out by the author's calculations showing a variation of practically 30 to 40% in stresses, and as a result of his analysis, he suggests three proposed engine loadings.

The writer has studied this matter carefully with the Chief Engineer of the Cincinnati, Indianapolis and Western Railroad Company, H. F. Passel, M. Am. Soc. C. E., and it seems that the first suggestion, namely, "the proposed engine loading (M-60)," would be the most desirable, for the reasons set forth by the author, that this diagram represents the least departure from present practice in bridge design, and that it more nearly represents graphically actual conditions.

G. H. GILBERT,† ESQ. (by letter).—The author is correct in the claim that the design of railway bridges for E-60 loading does not properly provide for the loading of modern types of engines and the actual axle loads used with such engines. A few years ago, the writer was requested to recommend a bridge loading for new specifications. Although no final recommendations were ever made, some preliminary data were developed, particularly as to the weight of engines in service on the railroads of the United States. The tentative conclusion was that modern locomotive loadings were properly protected by a typical Santa Fé engine with driver axle loads of 65 000 lb., assuming a 20-ft. driver wheel-base and a total wheel-base of 70 ft. of the engine and tender.

The author recommends a loading equivalent to E-75 for short bridges followed, for the longer structures, with a uniform train load of 6 000 lb. per lin. ft. Such a loading is equal to or greater than that of any locomotive now in service. It may be questioned whether railroad executives will approve of such a heavy loading. The Profession and railroad bridge engineers in general, can recommend only such heavy loadings as are in use to any appreciable extent, and such as are indicated by the trend of manufacture, as shown by locomotives built.

As indicated previously, it is considered that an E-60 loading does not properly protect the drivers of Santa Fé engines, nor properly care for the heavy

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† Engr. of Bridges, Southern Ry. System, Lines West, Cincinnati, Ohio.

axle loads now being used. Attempts to emphasize such facts are usually met with the claim that the heavy driver axle loads are exceptional and required by the use of special experimental engines or engines designed for special and particular operating requirements, together with the further claim that the Santa Fé type is not necessarily a permanent type and no great number of such engines are in service.

In Table 6, statistics are given, compiled largely from the *Railway Age Gazette,* showing locomotives for use in the United States, as ordered for each year from 1911 to 1921, inclusive. Table 7 gives the variation in weight of the several types of heavy engines, together with the approximate average loading per driver axle. These data would indicate that most of the heavy engines in service have driver axle loads between 55 000 and 65 000 lb.

TABLE 6.—LOCOMOTIVES FOR USE IN THE UNITED STATES.
(ORDERED 1911 TO 1917, INCLUSIVE.)

Type.	1911.	1912.	1913.	1914.	1915.	1916.	1917.
Mallet	112	168	72	59	120	218	175
Mikado	590	1 309	796	333	562	754	834
Santa Fé	...	...	...	63	75	325	370
Other types	2 148	3 038	2 599	810	816	1 594	1 345
Total	2 850	4 515	3 467	1 265	1 573	2 891	2 704

(ORDERED 1918 TO 1921, INCLUSIVE.)

Type.	1918.	1919.	1920.	1921.
Mallet	303	1	66	1
Mikado	1 001	11	634	81
Santa Fé	433	...	207	60
Mountain	53	...	69	25
Other types	865	202	795	43
Total for railroad	2 555	214	1 771	210
Total for industries	38	...	227	29
Total	2 593	214	1 998	239

(SUMMARY, 1918 TO 1921, INCLUSIVE.)

Type.	Number.	Percentage.
Mallet	271	5.7
Mikado	1 727	36.4
Santa Fé	700	14.7
Mountain	147	3.1
Other types	1 905	40.1
Total for railroad	4 750	100.0

Table 8 contains data, compiled from special articles, showing heavy driver axle loads. It will be noted that an average driver axle load exceeding 60 000 lb. is common, that this loading is as high as 67 000 lb. for three different

orders, and that, in one case, it is as high as 72 400 lb. The last loading and the highest single axle loading of 78 000 lb., are for a single engine.

TABLE 7.—WEIGHTS OF LOCOMOTIVES ORDERED FOR USE IN THE UNITED STATES.

Year ordered.	SANTA FE, 2-10-2.		MALLET, 2-6-6-2.		MALLET, 2-8-8-2.		MALLET, 2-8-8-0	
	Total.	Average driver axle (approx.)	Total.	Average driver axle (approx.)	Total.	Average driver axle (approx.)	Total.	Average driver axle (approx.)
1917.	321 000–411 000	52 000–65 000	420 000–429 000	58 000–59 000	465 000–548 000	51 000–60 000	.....	.....
1918.	352 000–390 000	58 000–61 000	427 000–440 000	59 000–61 000	540 000–	59 000–	.....	.....
1920.	361 200–383 000	59 000–60 000	364 000–441 000	49 000–61 000	476 000–494 500	52 000–55 000	494 500	58 500
1921.	378 000–394 100	60 000–62 000	.....	.....	.....	.....	505 000	58 000

The recent tendency of locomotive design is toward the increasing use of the Mikado and the Santa Fé types of engine, with little change as regards the use of Mallet engines. The heavy Mountain type of passenger engine is more in demand. This type is virtually an adaptation of the Mikado type for passenger service. On account of the extended wheel-base, its loading, from the standpoint of bridge design, may be neglected, as it will be lighter than the Mikado engine. During the last four years, the data presented show that 15% of the engines ordered are of the Santa Fé type and that 60% are strictly heavy power, with driver axle loads of from 55 000 to 65 000 lb.

TABLE 8.—HEAVY DRIVER AXLE LOADS.

Type of locomotive.	Average driver axle load, in pounds.	Highest driver axle load, in pounds.	Driver wheel-base.	Authority.
2-10-2.....	62 800	.....	22 ft. 10 in.	R. A. G., 12-7-17.
2-10-2.....	67 400	68 800	22 ft. 0 in.	A. R. E. A., *Bulletin No. 223.*
2-10-2.....	58 600	60 000	20 ft. 7 in.	C. N. O. & T. P. Ry.
2-10-2.....	67 500	.....	22 ft 6 in.	R. A. G., 8-3-17.
2-10-2.....	72 400	78 000	22 ft. 2 in.	A. R. E. A., *Bulletin No. 223.*
2-10-0.....	66 900	.....	22 ft. 8 in.	R. A. G., 6-15-17.
0-8-8-0.....	58 300	59 400	40 ft. 3½ in.	A. R. E. A., *Bulletin No. 223.*
2-8-8-0.....	57 800	.....	41 ft. 2 in.	R. A. G., 7-28-16.
2-8-8-2.....	59 000	.....	42 ft. 1 in.	R. A. G., 7-12-18.
2-8-8-2.....	60 100	61 800	42 ft. 1 in.	A. R. E. A., *Bulletin No. 223.*
2-8-8-2.....	58 400	.....	42 ft. 0 in.	R. A. G., 8-15-19.
2-8-2.....	63 800	65 500	16 ft. 0 in.	L. & N. R. R.
2-8-2.....	63 000	61 800	16 ft. 6 in.	A. R. E. A., *Bulletin No. 223.*
2-10-10-2.....	60 000	60 000	50 ft. 5 in.	A. R. E. A., *Bulletin No. 223.*
2-8-8-8-4.....	60 500	.....	67 ft. 7 in.	R. A. G., 1-26-17.

It appears obvious to the writer that, in selecting a proper bridge loading, the Santa Fé type can no more be ignored than any other type of engine. If that fact is once granted, it follows that many short spans and the floor systems of the through spans designed for E-60 loading will, at unit stresses, only take the Santa Fé engines when the driver axle loads are reduced to

about 53 000 or 54 000 lb. The tendency of driver axle loading is the reverse. A railroad using 60 000-lb. axle loads, with the old type of engines, such as the Consolidation, is forced by the need of tractive power to use 63 000 to 66 000 lb. for axle loads of its Mikado and Santa Fé engines.

In considering a broad matter like bridge building, affecting the development of railroads for the next 15 or 20 years, it is well to keep close to fundamentals. The types and weights which have developed within the last 10 years are due to an ever-increasing demand for tractive power. For passenger engines and freight engines of the Mikado type, there is the added demand of speed. The Santa Fé and Mallet types are probably restricted to use with drag freights, and the Mikado engine is the most popular freight engine. The use of a trailer develops the full value of the weight on the four drivers, so that in the future any increased tractive power must be obtained either by increasing the weights on the drivers, or by changing the type of engine. From some standpoints, the change to a Santa Fé or Mallet type is objectionable, so that fundamentally it seems reasonable that during the next few years there will be an increase in the weight of driver axles on the Mikado type of engine. Some railroads already are using about 64 000 lb. per driver.

To obtain tractive power greater than that afforded by the Mikado, it is necessary to use more driver axles. This leads immediately to the 5-axle or Santa Fé engine, or the Mallet engine with its 6, 7, 8, or even 10 axles.

The extended wheel-base used with Mallet engines greatly reduces the bridge loading when compared with the E-60 loading. For floor systems and short spans, there is no such reduction with the 5-driver axle, or Santa Fé type.

In studying this matter a few years ago, the writer compiled the diagrams, shown on Fig. 15, which compare moment, shear, and floor-beam reactions for the following locomotives:

1.—A U. S. Railroad Administration, heavy, Mallet engine of the 2-8-8-2 type, assumed as representing about the heaviest Mallet engine that would be used, except at special locations to meet peculiar operating requirements.
2.—A Denver and Rio Grande Santa Fé engine weighing 428 500 lb. This was assumed as typical of the heaviest Santa Fé engines built. It should be noted that the wheel-base is somewhat extended.
3.—A typical Santa Fé engine with condensed wheel-base and 65 000 lb. per driver axle.
4.—A typical Santa Fé engine with condensed wheel-base and 60 000 lb. per driver axle.
5.—An E-60 loading.
6.—A special engine of the Pennsylvania Railroad, Santa Fé type, weighing 442 840 lb.

The plotting of data for the last engine was of no practical value. It was interesting as showing the percentage by which it exceeds other loadings more generally in use. These diagrams (Fig. 15), support the statement that for spans in excess of about 100 ft., an E-60 loading is about heavy enough, as far as trusses are concerned, regardless of type of engine or weight of drivers. However, these diagrams also show that for a concentration of five axles on a 20-ft. wheel-base, with 65 000 lb. per axle, the loading demand for floor sys-

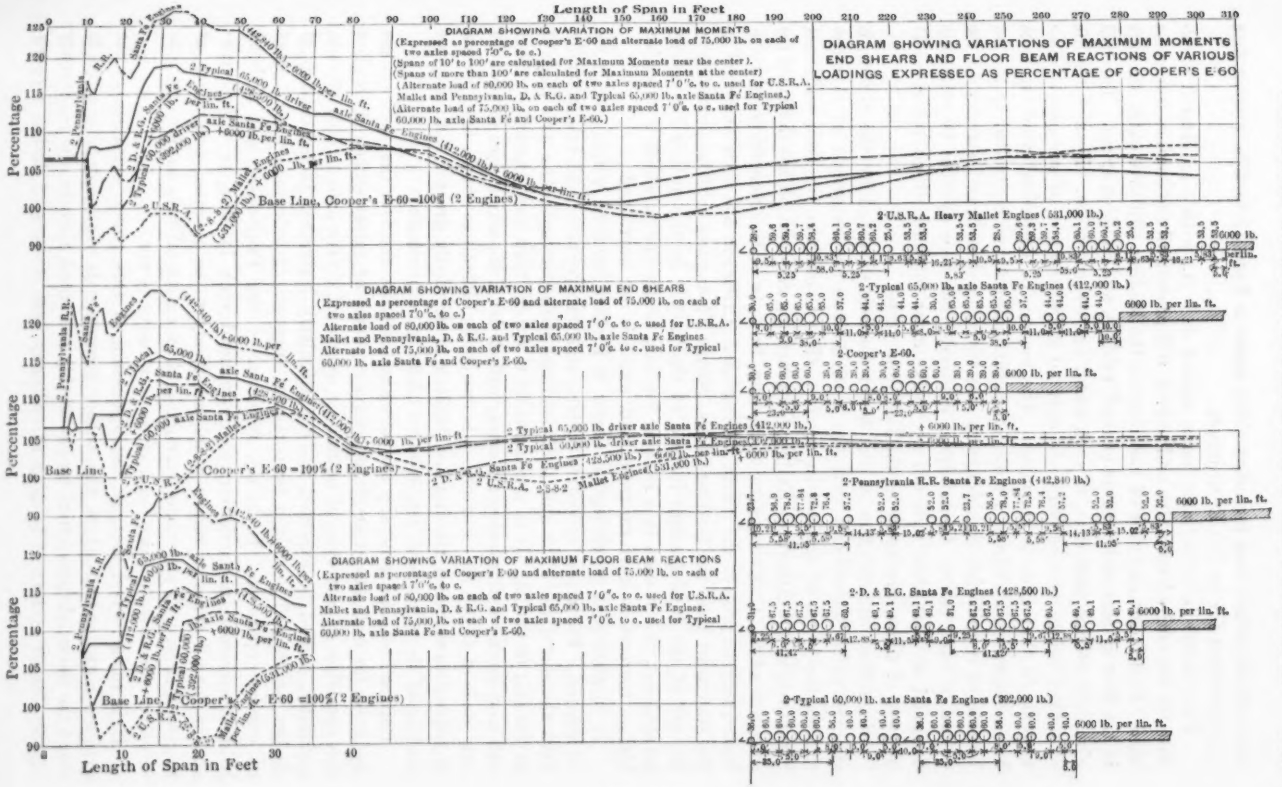

Length of Span in Feet
DIAGRAM SHOWING VARIATION OF MAXIMUM MOMENTS
(Expressed as percentage of Cooper's E-60 and alternate load of 75,000 lb. on each of two axles spaced 7'0" c. to c.)
(Spans of 10' to 100' are calculated for Maximum Moments near the center).
(Spans of more than 100' are calculated for Maximum Moments at the center)
(Alternate load of 80,000 lb. on each of two axles spaced 7'0" c. to c. used for U.S.R.A. Mallet and Pennsylvania, D. & R.G. and Typical 65,000 lb. axle Santa Fé Engines.)
(Alternate load of 75,000 lb. on each of two axles spaced 7'0" c. to c. used for Typical 60,000 lb. axle Santa Fe and Cooper's E-60.)
DIAGRAM SHOWING VARIATIONS OF MAXIMUM MOMENTS END SHEARS AND FLOOR BEAM REACTIONS OF VARIOUS LOADINGS EXPRESSED AS PERCENTAGE OF COOPER'S E-60
Percentage
Base Line, Cooper's E-60 = 100% (2 Engines)
DIAGRAM SHOWING VARIATION OF MAXIMUM END SHEARS
(Expressed as percentage of Cooper's E-60 and alternate load of 75,000 lb. on each of two axles spaced 7'0" c. to c.)
Alternate load of 80,000 lb. on each of two axles spaced 7'0" c. to c. used for U.S.R.A. Mallet and Pennsylvania, D. & R.G. and Typical 65,000 lb. axle Santa Fé Engines
Alternate load of 75,000 lb. on each of two axles spaced 7'0" c. to c. used for Typical 60,000 lb. axle Santa Fe and Cooper's E-60.
DIAGRAM SHOWING VARIATION OF MAXIMUM FLOOR BEAM REACTIONS
(Expressed as percentage of Cooper's E-60 and alternate load of 75,000 lb. on each of two axles spaced 7'0" c. to c.
Alternate load of 80,000 lb. on each of two axles spaced 7'0" c. to c. used for U.S.R.A. Mallet and Pennsylvania, D. & R.G. and Typical 65,000 lb. axle Santa Fe Engines.
Alternate load of 75,000 lb. on each of two axles spaced 7'0" c. to c. used for Typical 60,000 lb. axle Santa Fe and Cooper's E-60.
2-U.S.R.A. Heavy Mallet Engines (531,000 lb.)
2-Typical 65,000 lb. axle Santa Fe Engines (412,000 lb.)
2-Cooper's E-60.
2-Typical 65,000 lb. driver axle Santa Fé Engines (412,000 lb.)
2-Pennsylvania R.R. Santa Fe Engines (442,340 lb.)
2-D. & R.G. Santa Fe Engines (428,500 lb.)
2-Typical 60,000 lb. axle Santa Fe Engines (392,000 lb.)
6000 lb. per lin. ft.

FIG. 15.

tems of all bridges and for many short spans is from 15 to 20% greater than that provided for by the E-60 design.

Five 65 000-lb. driver axle loads, spaced at 5 ft., with a condensed wheel-base for the entire locomotive and tender, will protect all the engines commonly used. Under certain types of locomotives, the loading will give extra strength in the floor system and hangers of truss bridges, and in short spans. This will give only needed protection to such metal against corrosion. Even if the life of a steel bridge is assumed to be only 30 years, the loss of strength of floor systems in that time is alarming. The menace of corrosion, resulting from brine drippings, and from a policy of paint maintenance controlled by the net income of a railroad, would justify the use of reduced unit stresses in the design of such parts. Certainly, it demands that floor systems be originally designed so that they will not be weak under engines which represent 15 to 25% of those purchased in the last four years.

Regardless of criticism of the heavy loading proposed by the author, it appears to be self-evident that the E-60 designs are outgrown as a protection to the types of engines in service. The Committee on Iron and Steel Structures of the American Railway Engineering Association has defended the retention of the Cooper system of loading, in spite of representations as to its inadequacy to protect the Santa Fé engines. Criticism has been answered by the statement that the loading of the Santa Fé engines will be protected by the use of the E-65 loading instead of the E-60. As the author has clearly pointed out, this results in a loading for long bridge spans, which is excessive, and an increase in uniform train load, which is not warranted.

A fundamental psychological factor is further involved which should not be ignored. For years, executive officers, and transportation officials generally, have associated the E-50 and E-60 loadings with corresponding axle loads of 50 000 and 60 000 lb. It is unreasonable to expect that such executive officers will understand and approve a loading called the E-65, which does not protect axle loads of 65 000 lb. Further, it is extremely difficult to obtain the approval of expenditures when the implied loading is decidedly greater than the axle loading of the engines in use, or those likely to be purchased.

Railroad officials generally are being educated to the fact that new bridges designed for ordinary unit stresses have a factor of safety sufficient to allow the operation of any type of engine or weight of driver axle. Such officials, naturally, cannot be expected to be particularly interested in consistency of design which will permit a proportional increase of axle load for all types of engines operated. It is the function of the Profession, and railroad bridge engineers in particular, to recommend such loading as will be consistent for all types of power which are likely to be operated and which, at the same time, will appeal to financial and executive officials as properly conservative and based on axle loads of modern heavy power.

It is to be hoped that, in the discussion of this paper, criticism of the extreme high loading which the author recommends shall not be allowed to cloud the essential facts which he has so ably presented, namely, the inadequacy of the Cooper system of loading at ordinary unit stresses of design properly

to care for the present engine loading and for such engine loading as may reasonably be expected in the next five or ten years.

J. A. L. WADDELL,* M. AM. SOC. C. E. (by letter).—During the last four decades, the evolution in America in bridge designing has been marked from time to time by certain epoch-making papers, usually presented to the Society and thoroughly discussed by its members. To these papers and their discussions is mainly due the development of bridgework from a crude art to an advanced science.

This paper is worthy of being included in the list of such epoch-making papers. The Society and the Profession are deeply indebted to the author for his praiseworthy altruism in undertaking such an important investigation. The writer, for the last forty years, has been making somewhat similar studies. As long ago as 1892, in a paper entitled, "Some Disputed Points in Railway Bridge Designing"†, he treated at considerable length the subjects of railway bridge loadings and equivalent uniform live loads. Following the publication of that paper, he extended his investigations, and, later, wrote a pamphlet entitled "The Compromise Standard System of Live Loads for Railway Bridges and the Equivalents for Same." This pamphlet evoked much discussion in the technical press, as some engineers were opposed to the use of equivalent loadings, deeming exactness of stresses to be a *sine qua non*, and seeming to forget that all assumed loadings are merely typical and that they will never be more than approximately realized.

The result of the original papers and the ensuing numerous (and often quite warm) discussions was eventually to bring into general use the simple and economic method of calculating live load stresses in bridges by the use of equivalent uniform loads. In those days, that method was much more accurate than it is at present, for three reasons:

First.—Both the maximum and the average span lengths were then much shorter than they are now.

Second.—The weight per linear foot of the locomotives with their tenders did not then greatly exceed the weight per linear foot of the loaded cars, whereas, to-day, the excess is 50% or more.

Third.—Modern locomotives are longer than those in use three decades ago.

It goes without saying that the greater the excess of locomotive weight, as compared with that of a corresponding length of loaded cars, the greater will be the variations of stresses computed by the two systems—wheel loadings and equivalent uniform loads.

Comparing Mr. Steinman's investigation with that of the writer previously mentioned, there is as great a variation between them, in respect both to importance and science of treatment, as there is in the weights of present-day locomotives and those of that time.

The author has shown that the "Cooper" loadings should be relegated to oblivion; and, in the writer's opinion, the proposed new system is as good as

* Cons. Engr., New York City.

† *Transactions*, Am. Soc. C. E., Vol. XXVI (1892), p. 77.

can be evolved for the purpose of bridge designing. The argument suggested as a reason for retaining the Cooper loading, namely, "that it affords a convenient standard for rating bridges", is unsound, because the author's M-60 loading, with its proportionate M-10 unit loading, provides just as convenient a rating standard.

The "rating standard" evolved by Charles F. Loweth, President, Am. Soc. C. E., deserves to be considered as one of the "epoch-making" investigations in the evolution of bridgework, notwithstanding the fact that it is disassociated from designing.

Concerning the author's three suggested loadings, the writer is opposed to Loading No. 2 as being unscientific and illogical. It has no *raison d'être*, for it is much more troublesome to use than the equivalent-uniform-load method, and is but little, if any, more accurate.

As for Loading No. 1, it is well to retain it and include it in the specifications as the typical wheel loading, but it should be clearly stated therein that it is not to be used in computing stresses.

Loading No. 3 is by far the best, and should be generally adopted. As stated by the author, it is "the most scientific in principle and most convenient in application." In the writer's opinion, this claim cannot be gainsaid.

The specifying of a live load shown by wheel diagram and using instead a loading indicated by a formula that agrees therewith only approximately, may be objectionable to some engineers; but what does it matter if there is an occasional difference of a few per cent. between the two methods when both methods are merely arbitrary assumptions? To overcome this objection and satisfy the quibblers, let the formula be called the true standard and the wheel diagram its approximate graphic representation.

The thoroughness of the author's investigation and the clearness of his presentation of findings mark this paper as one of the best ever submitted to the Society.

GUSTAV LINDENTHAL,* M. AM. SOC. C. E. (by letter).—The author's painstaking analysis of the different locomotive loadings in use for the design of American railroad bridges is a valuable contribution to a subject which has been discussed for a long time, but which is not as vexing as is usually assumed.

The Cooper standard has become conventional, because it is sufficiently inclusive of maximum effects from concentrated loadings of locomotives and the grouping of such loadings. When locomotives become heavier, it is convenient to assume heavier axle loads without disturbing the grouping. Some locomotive axle loads have already reached nearly 80 000 lb. and some groupings of wheel loads, 370 000 lb. in 22 ft. There may be further increases in loads and changes in wheel spacing that cannot be foreseen. At times, bridges have to carry whole trains of dead locomotives. Meticulous accuracy, therefore, is out of place.

The author's proposed first method, substituting a new and simpler wheel-load diagram for the Cooper diagram, seems preferable to his other two pro-

* Cons. Engr., New York City.

posed methods. It would represent the smallest departure from present methods and is sufficiently inclusive of the maximum effects of loads from any of the seven locomotive-wheel diagrams illustrated in his paper. The Cooper diagram should also be illustrated in this paper, so that bridge engineers of other countries may know without searching elsewhere what the Cooper E-60 loading represents. Trusses in longer spans are less affected by the heavier locomotives than those of shorter spans, as may also be seen from Figs. 6 and 9.

The floor construction is merely a succession of short spans, therefore, it suffers more than the trusses from an increase of locomotive weights. The larger allowance for impact in floor members covers, to a certain extent, unforeseen increases in wheel loads.

Fast trains which produce the largest impact stresses, usually have lighter locomotives. The heavy locomotives considered by the author are used only with freight trains, usually at only one-half the speed of passenger trains, therefore, smaller impact stresses are produced. It is probable that heavy axle loads plus impact produce smaller stresses than the lighter passenger locomotives plus impact, at 50 to 60 miles per hour.

R. A. Caughey,* M. Am. Soc. C. E. (by letter).—The writer believes that Mr. Steinman has made a wonderful contribution to the Profession in the preparation of his paper and has done a great deal toward making changes in loadings for railway bridges, which many engineers have recognized as necessary, but have been unable to suggest on account of the enormous amount of work involved. He has proved that the Cooper loading should be replaced by one more in accordance with the weights and axle arrangements of modern railway rolling stock. He has also developed an equivalent uniform load which is really "equivalent", as he has taken into account the position of the section at which shear or moment is desired. Perhaps, it would be more accurate to say that three systems of equivalent loading have been developed, giving design results far more satisfactory than those of the Cooper system.

As to which of these systems it would be best to adopt, the writer would favor the first, the M-engine diagram, and might be criticized for doing so, as it would seem that either of the last two loadings would be easier to apply. This would be true under ordinary circumstances, but with a chart available, such as is given by the author on Plate X, and a table of equivalent uniform loads, such as that given in Fig. 2, no one who dislikes the long computations involved in stresses due to wheel loads would be forced to make them.

In many cases, the position of the engine loading (even though hypothetical) on the structure is desired and of the three schemes suggested the first is the only one which would give this. One case in which such information would be desired would be that of a complicated structure having a broken influence line some segments of which would represent positive and others negative stress and in which the maximum stress would have to be determined by trial positions of the engine and train.

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Another point in favor of the engine diagram, the writer believes, is that it keeps the computer more in touch with the principles involved in the computation of stresses due to moving loads; and it is his belief that the engineer who computes stresses without an understanding of the underlying principles is of little use to his employer. If the first method was in vogue, any computer would have the equivalent load as a time-saver and still not lose sight of the loading producing it.

The officials of some roads prefer to use their own engine diagrams in the design of bridges, and no doubt they would find the engine diagram closer to the conditions with which they deal and could use it best as a basis of comparison with their loadings. In such cases, the engine diagram would be the basis for "short cuts" such as equivalent uniform load charts and tables, and it would seem that it should be and could be so in the case of the M-loadings.

To sum up, it is the writer's belief that a change in bridge loadings should be made and that a simple, tangible system should be adopted. Much could be said for the simplicity of the third method, but it is a question as to whether railway bridge designers would care to think of a bridge loading as a thing so intangible as to be represented by an equation, and it may also be noticed on investigation that, for certain conditions, there is considerable variation from the "composite" loading when the equation is used. It seems that by the use of the author's charts, tables, etc., the first method will satisfy all requirements of simplicity and tangibility, will give a loading approximating most closely the "composite" loading, and, in addition, will be easy to use in bridge specifications.

While on the subject of railway bridge loadings, the writer wishes to state that, on branch roads, the controlling load in the design of bridges, especially short-span bridges, is the train load instead of the engine. To illustrate this, compute the maximum live load bending moment at the center of a 40-ft. plate-girder span, the traffic on which consists of an engine load followed by Pennsylvania railroad gondola cars of 220 000 lb. capacity, with an allowance of 10% for overload. The weight of these cars is 74 600 lb. It will be assumed that the engine is equivalent to Cooper's E-50. According to Table 3, a Cooper E-64 engine is equivalent to an M-50 loading when the center moment on a 40-ft. beam is considered. An E-50 engine, therefore, would be equivalent to $\frac{50}{64} \times 50 = 39$, say, M-40. The center moment due to a train of the coal cars mentioned will exceed nearly 15% that of the M-40 engine loading. From this, it would seem that attention should be given to this type of loading as well as to locomotive loading, especially when light locomotives are used.

J. C. Ralston,* M. Am. Soc. C. E. (by letter).—This paper embodies a comprehensive if not a conclusive re-cast of a question which heretofore has been in a state of general flux. As a comparative study, with a re-valuation of the essential factors, lighted by the last word of the mechanical or motive power engineer, and evaluated by the theory of probabilities in future loco-

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motive development, it is likely to clarify and standardize future specifications on loadings for many years.

Viewing the paper from the human standpoint of ethical ambition on the part of the author, it is expressive of the best devotional spirit of the Profession. Too often this spirit of contribution is lost sight of by the reviewer who would apply his own yard-stick in measuring engineering worth. Happily, this paper is timely, because it is serviceable in a practical way.

The redemption and the inspiration of the Profession lie in the contribution of a definite, usable, and needed technical asset. When such contributions are inspired by altruistic vision rather than profit, the genius of a great profession is finding its most lofty expression.

It is in this larger sense that the writer would commend the author. He has shown the erratics of the Cooper applications, or, as he states, "smoothed out the depressions," and given the bridge designer a more consistent and logical procedure for the determination of his controlling stresses.

It is safe to assume that the equivalent uniform loadings, applied either directly or through the author's so-called Composite, will stand for a long time.

The engineering offices of the motive power departments generally are turning their attention to the details of boiler and cylinder performances, to the metallurgy and the re-design of revolving and reciprocating parts, and to special devices, rather than to any further increase in or multiplication of axle concentrations. Road structures and clearances are putting a peremptory estoppel on further swelling the bulk dimensions of locomotives. Mechanical and energic improvements of locomotives have been delayed too long, but the obsession is now and will be toward carrying them to maximum finality.

The ratios between heating surfaces, grate areas, and cylinder volumes have been digested and re-digested carefully. At present, and hereafter, the mandatory is on that of providing a gas to produce the greatest cylinder horsepower and of obtaining greater calorific results per cubic foot of firebox volume. Pyramiding further axle concentrations on top of obsolete steaming practice is in limbo. These considerations being true, the railway bridge designer may look forward with confidence to an extended period for the application of the author's thesis.

Railway engineering has not been confined to the bold projects of territorial penetration, nor to the conquest of rugged and intransitable gorges, but covers also those masterful exploits which the patience, ingenuity, and genius of the mechanical engineer and the bridge engineer have accomplished. It has been an Augustan Age from the days of "Puffing Billy", the grass-hopper locomotive, to this hour of the super-Mallet and its concomitants, the colossal cantilever or the awe-inspiring arch. The "Rocket" locomotive of 1830, weighing 4½ tons, and the "Old Ironsides" of 1832, weighing 5½ tons, were feeble but epochal beginnings. Ten years later, the pace had been trebled. In 1842, Matthias Baldwin built for the Georgia Railroad, on order from J. Edgar Thomson, its Chief Engineer, fourteen of those famous "6-wheels-connected" locomotives. They weighed 12 tons each, or 4 tons per axle. Messrs. Thomson, Latrobe, and others of those days, and, later, Meigs, Fink, Bollman, Howe,

and their contemporaries were making great strides in bridge design. Yet, the work of each has become obsolete and inadequate practice. Newer and better engines and stronger and bigger bridges were demanded. Within the last decade, the bridge designer must draw his specifications for the requirements of locomotives with two and even three groups of drivers, each engaging four or five axles under concentrations of 60, 70, or 80 tons, and some ponderous machines weighed as much as 449 tons—one hundred times more than the original locomotive.

J. C. BLAND,* M. AM. SOC. C. E. (by letter).—In the Cooper specifications of 1890, the two heaviest loadings used were the "Lehigh Heavy Grade Engine Class" and "Class Extra Heavy A". Each loading consisted of two coupled Consolidation engines followed by a train designated as so many pounds per linear foot of track. Each engine was 32 ft. 7 in. long, with a tender 20 ft. 10 in. long.

The "Lehigh Heavy Grade" engine weighed as follows:

Pony truck	16 000 lb.
4 driver axles, at 40 000 lb.	160 000 "
2 tender axles, at 18 000 lb.	36 000 "
2 " " , at 20 000 lb.	40 000 "
Total	252 000 lb.

The train load was 4 000 lb. per lin. ft.

The "Class Extra Heavy A" weighed as follows:

Pony truck	16 000 lb.
4 driver axles, at 30 000 lb.	120 000 "
4 tender axles, at 18 000 lb.	72 000 "
Total	208 000 lb.

The train load was 3 000 lb. per lin. ft.

In 1894, in his paper entitled, "Train Loadings for Railway Bridges",† Cooper proposed the loading which since has been so generally used. This loading consisted of two coupled Consolidation engines, the length of the engine being 34 ft. and that of the tender, 22 ft. The train load was given as so many pounds per linear foot of track.

In this loading, the various axle loads were expressed in terms of the driver axle load considered as unity, that is, the pony truck axle load was 0.5; each of the four driver axles was 1.0; and each of the four tender axle loads was 0.65. Thus, the weight of the engine was 4.50 units; that of the tender 2.60 units, and the train weight per linear foot was 0.10 units. Cooper gave as a minimum the E-25, and as a maximum the E-40, loading. He seemed to have considered the latter the heaviest loading possible, and expressly said so regarding the train load of 4 000 lb. per lin. ft.

When the axle loads increased between the limits named, there was no change in the axle spacing, therefore, in computing moments, shear, etc., for

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† *Transactions*, Am. Soc. C. E., Vol. XXXI (1894), p. 174.

unit loads for this axle spacing, the finding of these functions for any E-loading was merely a matter of multiplication.

The advantages of this loading are many, but the disadvantages are:

(1).—That even in 1894, the ratio of tender axle load to driver axle load of 0.65 was smaller than that in actual engines; and the distance between the rear driver axle and front tender axle of 9 ft. was 30% less than in actual engines of the Consolidation type.

(2).—The weight of the train per linear foot was given as one-tenth the driver axle load, that is, the train load is a fixed proportion of the driver axle load.

(3).—The method was clearly inapplicable to engines of a type different from that assumed, namely, the Consolidation type.

As the weight of engines of the same type increases, the wheel-base and length increase, and although this is of not much account between the limits given by Cooper in 1894 for his proposed loading (namely, from E-25 to E-40), it becomes of consequence beyond E-40. In short, the E-loading is not applicable beyond Cooper's upper limit.

As to the train, there is no necessary relation between the weight of train per linear foot and the driver axle load, or, more generally, the weight on the driver axles. The gross weight of the train as a whole depends on the tractive power, and that again depends on the load on the drivers. Within the limits of the Cooper proposed loading of 1894 (E-25 to E-40), this did not make much difference, as E-40 engines were generally hauling 4 000-lb. trains, but the principle cannot be extended to E-60 engines, or greater.

In his paper, previously mentioned, Cooper wisely remarks that "the tendency to seek for a 'final standard' in any engineering construction, regardless of changing conditions, is a harmful one," and, by the same token, the writer considers that for many years the "Cooper loading" has ceased to be a standard. He concurs with the author when he states that "the time has come to make a fresh start, with a system based on the existing locomotive and train loads", and the writer feels confident, that if Mr. Cooper was still alive, he would share this view and be among the first to devise a new method and one which not only conformed to existing engines and train loads, but looked forward and beyond. The author's paper is so complete as to leave little room for discussion.

The writer studied the effects of some of the heaviest engines and train loads of the Pennsylvania System, and prepared a diagram which showed the "Cooper index" or "equivalent" for each foot of span up to 300 ft., for bending moments, shears, and cross-girder re-actions. The great variation of the "Index" for the same engine fully exhibits the inadequacy of the "Cooper loading" to represent the stress effects of modern heavy engines and trains. Table 9 gives the principal data for the heavy engines of the Pennsylvania System, as compared with the Cooper E-60.

Apparently, Cooper did not look forward to the use of engines other than those of the Consolidation (2-8-0) type, and the data given for the heaviest engine of that type used on the Pennsylvania System (the H-10-s), show the

great difference in many of the ratios, which, on the supposition of extending the Cooper system of loading beyond the E-40 given in his paper previously mentioned, should be the same.

The H-10-s engine has a Cooper equivalent or index of E-55 to E-50, and being of the Consolidation type, naturally shows less variation in the index than the engines of different types. Nevertheless, there are wide divergences in the ratios. The Cooper engine weighs 7.10 times the driver axle load, whereas, the H-10-s weighs 7.61 times the mean axle load. The Cooper tender weighs 2.60 times the driver axle load, whereas, the tender of the H-10-s weighs 3.18 times the mean driver axle load, or, expressed differently, the Cooper tender axle load is 65% of the driver axle load, whereas, the H-10-s tender axle load is 79.6% of the mean driver axle load.

It would be hardly fair to dwell on the divergences of the ratios of engines of other types from the Cooper ratios. One could scarcely expect to find a Consolidation engine of the Cooper type, which would produce the same stress effects as those caused by such an engine as the N-1-s; therefore, it is here that the author's suggested loading seems to fulfill the requirements. He makes a "composite" of all the heavy engines, and finds a loading which will come measurably close to the moments, shears, etc., caused by this "composite"; thus, all the irregularities of the actual engines are eliminated. Of the three forms of loading presented by Mr. Steinman for replacing the Cooper E-loading, the writer much prefers the third.

The Cooper loading has been of incalculable service, not indeed for the design of bridges, but for expressing the capacity of existing bridges. It has furnished a base, now fully understood by railroad men, whereby structures may be classified and compared. By taking an actual engine and determining the stresses in each member of an existing structure, and then by dividing each stress, moment, or shear, etc., by those due to the Cooper E-1, the Cooper index for each member is obtained, the lowest of which gives the capacity of the bridge. Any other typical engine would do as a base, but the Cooper engine serves the purpose far better, principally because of its general acceptance.

With the introduction, some years later, of engines with trailers, and also of engines of the Mallet type, the use of Consolidation engines for the design of bridges could hardly be maintained as good practice, and such railroads as had adopted the Cooper loadings as their standard were soon compelled to use different Cooper loadings for the different parts of the same bridge, as is pointed out by the author. Such a practice is not desirable.

The defects of the Cooper loading for the design of bridges having been so clearly shown by Mr. Steinman, the writer hopes that one of the three methods presented will soon be adopted by the Profession. The writer earnestly desires the abandonment of the Cooper loading for the design of new bridges of a capacity greater than E-40, but he has no such wish with reference to its use in stating the maximum capacity of existing structures; in short, the Cooper loading should still be maintained for use as the base of reference.

TABLE 9.—COMPARISON OF PENNSYLVANIA SYSTEM HEAVY FREIGHT ENGINES.

	Consolidation. 2-8-0	Consolidation. 2-8-0	Mikado, 2-8-2	Santa Fe, 2-10-2	Santa Fe, 2-10-2	Decapod, 2-10-0	Mallet, 0-8-8-0	Mallet, 2-8-8-0
Class of engine	E-60	H-10-s	L-1-s	N-2-s	N-1-s	I-1-s	C-C-2-s	H-C-1-s
Type of engine	2-8-0	2-8-0	2-8-2	2-10-2	2-10-2	2-10-0	0-8-8-0	2-8-8-0
Weight of engine, in pounds	270 000.	255 800.	335 100.	380 000.	435 000.	371 000.	460 000.	619 200.
Weight of tender, in pounds	156 000.	184 000.	191 300.	207 800.	207 000.	182 400.	210 000.	244 800.
Total weight, engine and tender, in pounds	426 000.	439 800.	526 400.	587 800.	642 000.	553 400.	670 000.	864 000.
Length of engine, in feet	34.0	40.7	50.0	56.0	57.2	49.6	59.3	70.0
Length of tender, in feet	22.0	30.9	32.2	37.4	34.0	32.2	34.0	34.0
Total length, engine and tender, in feet	56.0	71.6	82.2	93.4	91.2	81.8	93.3	104.0
Weight of engine, in pounds per linear foot	7 941.	6 285.	6 702.	6 786.	7 605.	7 480.	7 757.	8 845.
Weight of tender, in pounds per linear foot	7 091.	5 954.	5 941.	5 557.	6 088.	5 664.	6 177.	7 200.
Weight of engine and tender, in pounds per linear foot	7 607.	6 142.	6 404.	6 298.	7 089.	6 765.	7 181.	8 308.
Weight on driver axles, in pounds	240 000.	231 800.	253 500.	293 000.	341 000.	327 000.	460 000.	583 000.
Number of driver axles, and diameter, in inches	4.	4-62	4-62	5-68	5-62	5-62	8-51	8-62
Mean weight on each driver axle, in pounds	60 000.	57 800.	63 400.	58 600.	68 200.	65 400.	57 500.	72 900.
Maximum weight on driver axle, in pounds	60 000.	62 000.	72 560.	60 600.	75 600.	68 000.	62 000.	86 400.
Tractive power, in pounds	.....	205	205	190	215	250	225	205
Boiler pressure, in pounds per square inch	60 000.	53 200.	61 500.	74 000.	85 000.	90 000.	91 000.	130 500.
Ratio of weight on drivers to tractive power	4.0	4.33	4.12	3.96	4.13	3.63	4.0	4.46
Ratio of tractive power to weight on drivers	0.25	0.231	0.243	0.253	0.242	0.275	0.250	0.224
Tender axle load, in pounds	39 000.	46 000.	47 825.	51 950.	51 750.	45 600.	52 500.	61 200.
Ratio of tender axle load to mean driver axle load	0.65	0.796	0.754	0.886	0.759	0.697	0.913	0.839
Ratio of length of tender to length of engine	0.647	0.7592	0.644	0.668	0.594	0.649	0.573	0.486
Ratio of length of engine to total length of engine and tender	0.607	0.568	0.608	0.60	0.616	0.606	0.636	0.673
Ratio of weight of engine to mean driver axle load	4.50	4.43	5.29	6.48	6.38	5.67	8.0	8.49
Ratio of weight of tender to mean driver axle load	2.60	3.18	3.02	3.55	3.04	2.79	3.65	3.36
Ratio of weight of engine and tender to mean driver axle load	7.10	7.61	8.30	10.03	9.41	8.46	11.65	11.85
Ratio of weight per linear foot of engine and tender to mean driver axle load	0.1268	0.1063	0.1010	0.1074	0.1032	0.1034	0.1249	0.1139
Ratio of weight of engine to tractive power	4.50	4.81	5.45	5.14	5.12	4.12	5.06	4.74
Ratio of weight of engine and tender to tractive power	7.10	8.27	8.56	7.94	7.55	6.15	7.36	6.62
H-21-cars, marked capacity in pounds	140 000.; overload 10%; weight 56 000 lb.; length 44 ft.; weight per linear foot of train = 4 773 lb.							
G-23-cars, marked capacity in pounds	220 000.; overload 10%; weight 86 000 lb.; length 54 ft.; weight per linear foot of train = 6 000 lb.							
Ratio of weight per foot, H-21-train, to mean driver axle load	0.0795	0.0826	0.0753	0.0814	0.0699	0.0729	0.0880	0.0655
Ratio of weight per foot, G-23-train, to mean driver axle load	0.10	0.1088	0.0946	0.1024	0.088	0.0917	0.1023	0.0823

NOTE.—Tractive power for mean effective pressure = 85% of the boiler pressure, except in cases of the I-1-s and the H-C-1-s. For the I-1-s, the tractive power = 75% of the boiler pressure, and for the H-C-1-s, it is based on weight on drivers.

With reference to the design of structures, the writer wishes that each railroad would consider the group of its engines for which it desires to provide, form a "composite" of such engines, and then choose the M-loading (taking into account future development) that best fits its case, and use such loading in its specifications.

"Taking into account future development," stated previously, presents a problem. Shall a given percentage be added to each of the engines forming the "composite"? Or, shall only those of the group which one thinks will in time be increased be considered? The easy way would be to increase each one of the group by a given percentage—the better way is to consider the question further. The Mikado engine is a Consolidation engine with an added axle—the trailer—which in a sense may be called non-productive weight, as it does not increase the adhesion of the drivers. In the I-1-s engine, however, the added axle is a driver, and, therefore, the adhesion is increased.

Comparing the L-1-s engine with its predecessor the H-10-s, the weight on the drivers was increased only 9½%, while the weight of the engine increased 32%, and the tractive power increased only 16 per cent. Comparing the I-1-s engine with the H-10-s, the weight on the drivers increased 41%, and the weight of the engine increased 45%, whereas, the tractive power increased 69 per cent. The I-1-s would seem a better and more economical development than the L-1-s.

Again, compare the N-1-s with the H-10-s. The weight on the drivers increased 47%; the weight of the engine increased 70%; and the tractive power increased 60 per cent. This does not seem as economical a development as the I-1-s, and although the I-1-s weighs 96% of the N-1-s, its tractive power is 6% greater, and the weight of engine per pound of tractive power is 4.12 as against 5.12 for the N-1-s; that is, a pound weight of the I-1-s is more productive than a pound weight of the N-1-s.

Thus, it would seem that the I-1-s type of engine is more likely to be increased in the future than the L-1-s or N-1-s types.

The stress effects of the I-1-s type do not differ much from those of the N-1-s; therefore, it is immaterial if both were increased in the same proportion. Prognosis as to engine development, however, is extremely hazardous, the writer merely presents the facts as they stand to-day and ventures a few comments.

J. M. Johnson,* M. Am. Soc. C. E. (by letter).—The author presents, in a convincing manner, the defects of the Cooper system of rating as applied to many of the present-day locomotives. The writer agrees with this indictment, if it is confined to those roads having a heavy and steady tonnage, consisting principally of coal, ore, and furnace products, requiring for its economical movement, under diverse conditions, the various locomotives (seven in number) tabulated by the author. For such roads, the Cooper rating fails, or is unsatisfactory, and could be replaced with advantage by the author's system, based on any one of the three proposed loadings.

* Marion, S. C.

There is, however, a large mileage in the South, Southwest, and parts of the West, constructed, of necessity, originally with a large curvature and with comparatively light rails. Also, from necessity, this condition is likely to remain practically unchanged for some time. On many of the roads in this territory, the heaviest power is of the Mikado and Mountain types of locomotives, the curves (shear and moment) of which agree quite closely with the Consolidation type of equal axle loads, and, in consequence, the Cooper loading gives satisfactory working results. On other roads of the same region, in addition to the Mikado and Mountain types, 2-10-2 locomotives with maximum axle loads of about 60 000 lb. are in use on divisions where the curves are not too sharp. The shear and moment curves of this engine, for equal axle loads, are somewhat above the Cooper curves, but the difference between the curves when only one 2-10-2 locomotive is used, is not marked, except for short spans. Engines heavier than the three types mentioned would have no place on these roads, except that occasionally a locomotive of the Mallet type is used as a pusher on a mountain grade.

C. D. Purdon, M. Am. Soc. C. E., investigated the relation between the Cooper loading, and various heavy locomotives, with axle loads averaging 55 000 lb. The result of this investigation is shown by curves, originally published in *Engineering News* of October 23, 1913, from which it is interesting to note the practical agreement of the Consolidation and Mikado curves, and the decided departure of the E-55 curve, from that of the Consolidation engine as usually built. The curve of the Decapod—the nearest to the general 2-10-2 engine—is above the Cooper diagram for girder spans, but below it for truss spans. A study of these curves leads to the conclusion that, for the Mikado and 2-10-2 engines, with axle loads of about 60 000 lb., the Cooper loading gives satisfactory results.

The writer believes that the loading suggested by the author is in the right direction, but thinks its application now to the entire country is too radical, inasmuch as there is a large mileage for which the Cooper loadings will be satisfactory for a long time.

C. S. G. Rogers,* Esq. (by letter).—The author of this interesting paper presents the inconsistencies of stress resulting from applying a conventional engine loading and wheel-base such as that of Cooper, to the design of bridges which are to carry engines of axle loadings and wheel spacings relatively so different from the conventional loading.

The new loading, M-60, appears amply to cover all stresses in all lengths of spans that may result from the use of five of the heaviest, present-day types of engines. If all these five types of locomotives were in use on any one railroad, the proposed conventional loading would admirably conform to the requirements. Is this the case, however? Is it not rather that each railroad appears to be developing its engines along lines peculiar to itself?

The force of the author's arguments, therefore, seems to point, not to a conventional engine loading that will prove satisfactory, no matter what type of locomotive is used, but rather to loadings that will specifically meet the

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type of locomotive peculiar to the railway under consideration. Unless this is done, the extra material put into the bridge structure to suit the requirements of engine loadings of foreign roads is a direct and needless waste and will never be called on to function.

It is noted, too, that although the main argument centers about the new conventional loading, M-60, other loadings proportional to the front group of drivers and to the uniform train load are recommended for those railways the locomotives of which are not of the size of the five heaviest under consideration.

The fact is, however, that where the engines used are of a less powerful type, they are not merely reduced models of these heavier types (2-8-8-8-2, 2-10-10-2, 2-10-2, etc.), but usually have different groupings of drivers. The problem, therefore, does not appear to be solved by adopting a loading lighter and regularly proportional to M-60, but rather in discovering a loading which will conform to the various locomotives in use or likely to be used on the special railway under consideration. No one of the five railways, the heavy locomotives of which have been considered in developing the M-60 loading, is concerned in making its bridges strong enough to carry the heavy locomotives of the other four railways.

Looking at the matter from another angle, it is shown that the Cooper E-60 loading nearly satisfies the stress requirements for these five heavy engines for the longer spans, yet would leave an over-stress of approximately 25% in some of the shorter spans. Conversely, should the engines in use on any railroad reasonably conform to any of the Cooper loadings (a not unreasonable supposition), the use of the M-60 loading (or one of a proportionate series) would result, in the shorter spans, in a waste of the 25% excess material that the M-60 loading would provide. Therefore, from this point of view, too, the adoption of the M-60 loading is not the real cure for the evils resulting from a loading that does not conform to the actual loading, and the problem still appears to be the discovery, on the part of each railroad, of a conventional loading to satisfy its special needs.

The author, however, has made a valuable contribution to the problem of bridge design in so forcibly calling to the attention of the bridge engineer the inconsistencies resulting from adopting an arbitrary engine-loading diagram without carefully analyzing, on a percentage basis, the relation between the stresses produced by the heavier or critical engines in actual use on the lines under his jurisdiction and those produced by the conventional engine, and correcting the latter before adoption.

E. A. Stone,* Esq. (by letter).—The author's reasons for classifying the Cooper system of loading among the relics of the past seem to be beyond dispute, and, therefore, his proposed new loading system is a long step in the direction of filling a recognized need.

The three solutions of this problem indicated in the paper probably cover, in general, all methods that might be brought forward. The first method, a wheel-load diagram, would be a continuation of present practice and, after having made up a table such as that shown in Fig. 2, it would be as simple to apply as any other method.

* Montreal, Que., Canada.

The author is entitled to the sincere thanks of all bridge engineers for the exhaustive manner in which he has presented this subject, particularly in the tables and diagrams.

Would it not be desirable and opportune, in view of the present state of bridge specification loadings, to appoint a committee to consider this matter fully and endorse the author's proposed M-60 loading, or bring forward some other loading which might be deemed preferable?

HAROLD C. BIRD,* ASSOC. M. AM. SOC. C. E. (by letter).—For more than thirty years the subject of concentrated wheel loads has been discussed. This paper, fortunately, comes at a time when members of the Society are interested in the adoption of a new specification for bridge design and construction.

When the late Theodore Cooper, M. Am. Soc. C. E., first specified his Class A and B as Consolidation locomotives, he probably did not anticipate that the Consolidation type of locomotive would be the type governing bridge design for so many years. Familiarity with the Cooper series should not prevent the adoption of another live load specification if the latter more nearly fits the needs. The idea of any load system specified is not to represent an actual engine or train, but to be an imaginary group of wheels (or its equivalent) which should give as great, if not greater, stresses than any which are likely to occur at present or in the immediate future. If the Cooper loading does this, and does it economically, there is no reason to make change.

The author has treated the subject in a thorough and enlightening manner which should convince those who will permit themselves to be convinced, that the Cooper loading does not carry out the idea expressed previously, nor does it fit the needs for rating bridges. The fact that there was difficulty in the shipping of some of the new locomotives is another indication that bridges are not prepared to carry the loads of the present time.

Although the writer has used Mr. Steinman's charts (similar to Plate IV) in the classroom for several years and has felt that they are satisfactory, he imagines that many engineers have never used them, and thinks it would be well if the author would illustrate the use of the charts with typical problems and show how they are plotted. The writer notices that the tables are not labeled with longer and shorter segments, which is an oversight.

The discussion† by F. E. Turneaure, M. Am. Soc. C. E., on the Specifications for Steel Railway Bridges submitted by the Special Committee on Specifications for Bridge Design and Construction, or the original,‡ should be referred to by all those studying this paper. The writer has calculated a number of cases using Mr. Steinman's proposed series and has been pleased to note how well the values fit the maximums of the moment and shear diagrams of the American Railway Engineering Association. The making of such diagrams and especially the calculation of shear and moment tables similar to those in Ketchum's "Structural Engineers' Handbook," would be a helpful addition to the paper.

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* Prof. of Civ. Eng., Pennsylvania Military Coll., Chester, Pa.

† See p. 532.

‡ *Proceedings*, Am. Ry. Eng. Assoc., Vol. XXI.

The paper will also serve somewhat as a stop to the arguments as to whether concentrated wheel loads, equivalent uniform, or a uniform load with excess or excesses should be used, for, here, engineers can have their choice, with results practically identical. This will give those who so desire, an opportunity to visualize and, in addition, will offer those who desire excess loading, an opportunity to have their way. The formula submitted should also merit a great deal of attention.

The idea suggested by Henry B. Seaman, M. Am. Soc. C. E., that live loads shall consist of either the Cooper or the Steinman series, as may be specified by the engineer, is a good one.

If nothing else is accomplished by this paper, engineers have been given an easy way to compare the stresses produced by the Cooper series with those produced by the heavy engines of to-day.

ALMON H. FULLER,* M. AM. SOC. C. E. (by letter).—It is difficult to foresee the far-reaching effect of this paper. The Cooper loading no longer represents even to an approximate degree the modern locomotives and trains; it is still used, first, because it has been used; second, because tables and diagrams are available for ready computations; and, third, because nothing better has been offered.

It now seems apparent that the third reason has been removed by the presentation of this paper. The second reason is also partly met by the tables and diagrams which have been presented by the author. Other tables, such as moments and shears for various panel lengths and numbers of panels for truss work and for moments, shears, and floor-beam reactions for girder spans, may be constructed, the work on which would be insignificant in comparison with its importance. The first reason has very little validity beyond that of natural inertia, and to persist in the use of this obsolete standard would be not only unscientific, but would result in uneconomical designs.

Of the three suggestions which Mr. Steinman has offered, one may likely have difficulty in making a choice. The writer would eliminate the second suggestion; although it has the merits claimed by the author, it lacks the definite picture presented with the first and the scientific background of the third suggestion. The writer has tried to get the viewpoint which evidently leads Mr. Steinman to prefer the third, but has yet been unable to do so to the extent of giving up the concrete picture presented by the first suggestion.

It is true that the formula would prove to be a "time saver" (for certain classes of work), that its scientific foundation yields curves of greater regularity and smoothness, and that it is independent of reference books to a certain extent. In these days, little designing or stress computation is done, or need be done, without access to an office library. When the tables and diagrams of the paper have been supplemented by the tables† for moments and shears adaptable to structures with panels presented to the Society by O. E. Selby, Jun. Am. Soc. C. E., and by girder tables for shears, moments, and floor-beam reactions, such as are now so generally available for the Cooper

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† *Transactions*, Am. Soc. C. E., Vol. XLII (1899), p. 223.

loading, the work of computation will be as simple for the engine diagram as for the formula. The diagram will be much more expressive than a formula on a drawing or in a specification. The fact that the formula gives a different uniform load for various parts of the same span, makes it necessary to refer to tables or diagrams for rapid work, and the use of the table or diagram will require as much time from one as from the other. Even though general use shall be based on the engine diagram, the formula, either in memory or in notebook, will be a convenient device for computations which must sometimes be made quickly and without office facilities. The author apparently had this in mind in referring to it as a "time saver." The writer wishes in no way to depreciate the ingenuity of Mr. Steinman in establishing this formula. He recognizes it as an asset in putting a finish on the entire discussion and in having a legitimate practical application as previously noted. He believes, however, that the greatest contribution from a practical standpoint lies in the first suggestion for a definite engine diagram.

After another half century or so, this diagram may represent existing loads no better than that of Cooper does at present. If that should be the case, the Profession, perhaps, will find it worth while to make another re-adjustment, and in doing so, this paper should prove to be of great assistance. If the author can show that the formula may be readily extended to rapidly changing situations, it would not only be an added reason for adopting the third suggestion, but, perhaps, the controlling one as well.

CARLTON T. BISHOP,* M. AM. SOC. C. E. (by letter).—The author has shown that the Cooper loading has outgrown its usefulness. It has served its purpose well, but the modern loadings have not increased proportionately, and some change should be made. A much simpler form of loading could be devised to give results as satisfactory as those which the Cooper loading now gives, but this would not be sufficient. Any new loading should give approximately the same results as the actual loads which are likely to be used in the near or distant future.

The author has made an exhaustive study of existing loads and of the trend toward the future. He has devised a composite loading which appears to be satisfactory, but, best of all, he has developed a formula for equivalent loads, which has great possibilities. This formula, or its corresponding tables or diagrams, can be used more advantageously than the concentrated loads and gives consistently good, if not better, results. No one system of concentrated loads can give precise stresses for different actual loads, and the simpler equivalent uniform load seems much superior. Mr. Steinman has blazed the way by proposing a formula for uniform loads in terms of segments of the span, thus overcoming the principal objections to the use of such loads, that have heretofore been advanced. The stresses obtained from these new equivalent loads provide for any of the critical engines of to-day, and, probably, by proportion, for those of years to come.

The writer hopes that future specifications will give the live load in terms of such a formula as that suggested by the author. Before such adoption,

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however, a committee should study the question and agree that the proposed formula is the best obtainable.

CLYDE W. MACCORNACK,* M. AM. SOC. C. E. (by letter).—The writer is much interested in this excellent paper which is one of the most valuable contributions to bridge engineering literature in the past twenty years. It has been generally recognized for some time that the Cooper loadings do not properly represent modern locomotive loadings. Pressure of daily work has no doubt prevented many engineers from making as careful an examination of this subject as they have wished, and they will welcome the data contained in the paper, which give facilities for making the necessary comparisons. It is to be hoped that, as a result of this paper, a more modern system of locomotive loading will be generally adopted.

A standard type of loading should represent within close limits the actual loads in use. The proposed wheel-load diagram appears to satisfy this condition. Methods of computing stresses by means of the engine diagram have been so fully developed that the labor involved is not an important consideration. The engine diagram, representing actual loads in service, should always be used in examining existing structures. It does not follow, however, that the engine diagram should necessarily be adopted for designing new bridges. Any system of loading which will give resulting stresses equivalent to those of locomotives in use, would appear to the writer fully to satisfy the requirements. If economical designs can be produced by a simpler loading, it should be substituted for the engine diagram.

It is of advantage to adopt a loading which would have the general characteristics of the actual loads on the structure. Bridge floors and short-span bridges without floors are subjected to direct loads from the locomotive drivers. In the writer's opinion, such parts and structures should be designed for the heaviest actual loads in service. The stresses in trusses and girder parts where the loads are applied through a floor system, reach the trusses and girders in the form of panel concentrations. This form of loading is as well represented by a uniform load as by a system of wheel loads. The writer believes that a system of wheel concentrations representing the heaviest loads in use should be adopted for bridge floors and for short spans without floors and that a uniform load should be used in designing all other bridge members. The computation of stresses for such a system of loading would be reduced to the simplest operations, and, at the same time, the visualization of the loadings would be retained in the designer's mind. A load formula such as that offered in the paper or, if preferred, modified to coincide more nearly with the composite loading, should give as satisfactory results as an engine diagram.

It is to be hoped that this paper will result in the adoption of a new standard system of loading representing modern locomotive loads. The important consideration is not the type of loading, whether it be an engine diagram or an equivalent uniform load formula, or any modification of these,

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but that the new loading be such that it will represent correctly conditions created by modern locomotives in all parts of the bridge structure.

C. P. DISNEY,* ESQ. (by letter).—Mr. Steinman is to be congratulated on having made such an exhaustive study of this subject. His proposals are practical and may be considered as a solution of the need for a system of calculating stresses in bridges, which is abreast of the present progressive development of locomotives and trains in the United States.

All three of the alternative proposals presented have distinctive advantages over present methods. The third proposal, however, seems to be ideal, as it is a formula which can be memorized and used, without tables or charts, to find the moments and shears at any point of a span by a simple slide-rule operation.

As regards the application of this proposed system of loading to Canadian railways, it would appear that, although Mr. Steinman's system could be modified to conform to locomotives and trains operated in Canada, the Santa Fé type is, the writer believes, about the limit, considering the capacity of road-beds and bridges, and this locomotive, with a train of 100-ton capacity cars, can still be taken care of by using the Cooper loading.

In a 200-ft., through truss span, the maximum difference between the stresses produced by the Cooper E-55 loading and the corresponding stresses produced by a double Santa Fé, with a 5 500-lb. following load, including the floor system, is only 5 per cent. The 5 500-lb. load can be considered as a train of 100-ton-capacity cars, which is the most severe probable condition of loading likely to occur on Canadian lines.

In conclusion, the writer would state that he considers Mr. Steinman's proposals to be in line with the advanced type of trains operated in the United States to-day, whereas the Cooper system conforms to the type of trains operated in Canada, which type seems likely to continue for many years.

O. H. AMMANN,† M. AM. SOC. C. E.—The loadings proposed by Mr. Steinman unquestionably have the advantage of greater simplicity over the Cooper loading. This advantage, however, would not be sufficient to justify the abandonment of a loading which has been widely adopted as a standard. Tables and diagrams for the Cooper loading, including those of the author for equivalent uniform loads, greatly simplify the calculation of stresses for that loading.

Examination of the elaborate data contained in the paper has not convinced the speaker that the Cooper engine loading is less representative of actual engines than that proposed by the author. On the contrary, he is more convinced than ever that it is futile to attempt to introduce a loading which embodies any marked improvement over that of Cooper.

Consider, first, the engines alone as affecting spans up to about 120 ft. The author's engine bears no greater similarity than does that of Cooper to any of the seven heaviest existing engines, not to mention the many other heavy engines, steam and electric, still in use, and those likely to be evolved in the future.

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The apparent improvement of the author's loading over the Cooper system is due to inconsistent presentation of the facts. The author condemns the Cooper loading on account of its discrepancies with existing individual heaviest engines, and emphasizes the close agreement between his M-60 loading and the composite of heaviest existing locomotives. He does not disclose the equally large discrepancies between the M-60 loading and individual heaviest existing engines, nor the equally close agreement between the Cooper loading and the composite of heaviest existing engines. All these facts are clearly shown on the two diagrams (Fig. 16), which are self-explanatory. These diagrams also show the small differences (only up to 6% for shears and 7% for moments) between the M-60 loading and its average equivalent, the Cooper E-72 loading.

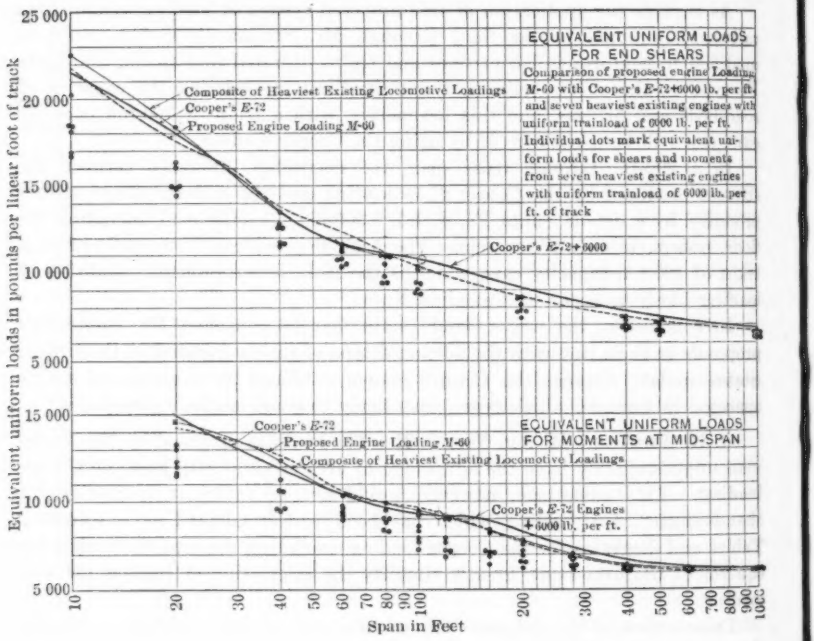


FIG. 16.

Considering the fact that differences between individual existing engines are several times larger, and also the many other uncertain factors, such as impact, wind, secondary stresses, quality of materials, etc., which factors, as variously specified, give, for the same live load, sections differing by as much as 40%, do these small differences between M-60 and E-72, even if they were improvements, justify the abandonment of an established loading?

Consider the combined engine and uniform train load: The apparent discrepancy between the Cooper loading and actual engines in short and long spans is not due to the Cooper engines, but to an inconsistent combination of

the engines and the uniform train load behind them. To represent the heaviest modern engines with a train load of 6 000 lb. per ft., as assumed by the author, and to compare equitably with the M-60 loading, the Cooper E-72 engines should be combined with the uniform train load of 6 000 lb. If that is done the apparent discrepancies of the Cooper loading and the existing heaviest loadings, for short and long spans disappear, as may be noted in Fig. 16. The Cooper curve is as smooth and follows as consistently the curve of composite heaviest existing loadings as the curve of the M-60 loading. Is there any guaranty that the weights of locomotives and cars will not continue to increase at different rates? If that should be the case, the author's loading would become as inconsistent as that of Cooper, unless its axle concentrations were also changed in a ratio different from that of its uniform train load. It is impossible to have an ideal loading for all conditions and all times. Loadings must be changed from time to time, but such changes should be made with the least possible disturbance of the generally adopted standard, unless commensurate advantages can be gained otherwise.

The speaker would recommend the retention of the Cooper loading, but would leave the variation of the engine concentrations and the uniform train load by suitable, not necessarily the same, ratios to the discretion of the engineer. Such a specification would be sufficiently elastic to cover practically any present or future loadings. This suggestion is not new, a different ratio for engine concentrations and uniform train load has already been used, notably in the design of the Metropolis Bridge.

The speaker would favor a formula for uniform load, except for the fact that such a formula does not convey to the mind a conception of the loading which it represents, and which is desirable for general comparison with actual engines. The author deserves much credit for his elaborate investigation without which proper judgment of this question would hardly be feasible.

HENRY B. SEAMAN,* M. AM. SOC. C. E.—The speaker wishes to express his appreciation of this paper. To have the subject presented in such detail is a great satisfaction.

The next question is its practical application and introduction. Bridges generally, throughout the United States, are rated on the Cooper E-system; all the records are on that basis, and to make such a radical change would hardly seem justifiable. Therefore, a double specification, providing for a multiple of either the E-10 or the M-10 loading, whichever may be preferred by the designing engineer, has been proposed by the speaker.

It is believed that future heavy bridges will be designed for the M-loading. It will not be difficult to alter the classified records of the railroads if the engineer so desires, as tables are furnished for that purpose. Mr. Steinman is to be congratulated on the results of his studies.

CHARLES A. MEAD,† M. AM. SOC. C. E.—The necessity for revision of the live loading for railroad bridges is becoming more and more apparent. Every

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one who has investigated the stress-producing effects of actual modern engine loads on existing bridges that were designed for the Cooper loading, must be impressed with the lack of uniformity of stress throughout the various parts of the structure. This has resulted in wasted metal in some parts and insufficient section in others, the differences assuming a magnitude beyond the E-50 class, which makes imperative some changes in loading specifications to fit modern practice.

It is evident that train loads and locomotive loads do not vary in the same ratio. Train loads in general use are fairly well fixed, owing to the interchange of cars, so that comparatively unimportant branch lines and secondary railroads, with light motive power, will be subjected to the same train loading as the main lines with their heavy power. At present, train loads do not exceed 6 000 lb. per lin. ft. of single track, except in a few instances which will not affect the conclusions reached. Furthermore, the present design of cars must be changed so radically, in order to increase this train loading, that it seems as if this uniform load would represent a maximum train loading for an indefinite period.

Locomotive loadings do vary greatly on different roads and on the several sections of the same road, and necessarily so for economic reasons. Therefore, it is inconsistent that locomotive and train loads should be considered to vary proportionately. Mr. Steinman has gone a long way toward adjusting this incongruity. He has also produced a simple specification requiring a minimum amount of work in computation.

Good bridges can be designed for a uniform load plus spot loads, as suggested in Mr. Steinman's second series. This method is not new, but it has been presented in this paper more comprehensively than ever before. The selection and spacing of spot loads can be adjusted to suit the loading requirements of various railroads, resulting in more economical bridges. From its simplicity and its adaptability to various conditions, the second loading method is especially attractive.

HERBERT C. KEITH,* M. AM. SOC. C. E.—The author is to be congratulated on his paper; but bridge engineers are more to be congratulated on his having presented to them in this paper the results of his studies.

In 1882, under the direction of the late Samuel E. Tinkham, M. Am. Soc. C. E., the New York and New England Railroad Company adopted bridge specifications with a live load headed by a consolidation locomotive—a new type at that time—with a driving wheel load of 24 000 lb. per axle, or 96 000 lb. total weight on the drivers. A short time afterward, Mr. Tinkham thought it advisable to increase the engine load to 25 000 lb. per driving axle, or 100 000 lb. on all the drivers.

When it is noted that one of the engines mentioned by the author has a load of 300 000 lb. on the drivers, and another has an axle load of 78 000 lb., it is amusing to one who remembers the criticisms that were made at the time Mr. Tinkham and the New York and New England Railroad Company saw

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fit to increase the weight on the drivers, in their specifications, to 25 000 lb. per axle and 100 000 lb. total driver load. At that time, many railroad men said that such a heavy engine would never be built; and as the New York and New England Railroad Company was not very prosperous in those days, it seemed somewhat ridiculous to them that such a poverty-stricken railroad, should use such heavy loads. Within three years that road had engines in use heavier than those that had been considered as excessive. The changes of the future cannot always be foreseen. There have been periods when the weights of locomotives have increased, and these periods have been followed by a slackening in the rate of increase of locomotive loads, during which the weights of cars have been increased, as when the large high-sided coal cars came into use, until now an equivalent to 6 000 lb. per ft. is a commonly accepted train load.

A few years after Mr. Tinkham left the New York and New England Railroad Company, a serious accident occurred near Boston, Mass.—Bussey Bridge collapsed under the weight of a train. This accident caused an investigation to be made by the company of the condition and capacity of its bridges, and the speaker was appointed to that task. He first studied the various engines used on the road, especially the heaviest of each type; there were Consolidation engines, Moguls, and the American type with only two driving axles. It was found that in bridges carrying all these engines, the maximum stresses in different members was caused by different types of engines; that is, one member might receive its maximum stress from a Consolidation engine, whereas some other member would be stressed more by a Mogul or an American engine. In some cases a member would receive its maximum stress from a lighter, but more compact, engine, than from the heavier, but longer, engine of the same type. It would have been quite a task to make calculations for all the bridges for all these locomotives; therefore, after some study, it was decided to use as a basic load for this investigation, a uniform load of 4 000 lb. per lin. ft., which was the recognized heavy train load at that time, with an excess load of 3 000 lb. per lin. ft. for a length of 25 ft. For the purpose of comparing the effect of the different locomotives, tables were prepared, that gave, for different lengths of span and different positions in the span, the relative moments and shears of various heavy locomotives that were used on different parts of the road, as compared with the basic loading of 4 000 lb. per lin. ft., with an excess load of 3 000 lb. per ft. for a length of 25 ft. This loading was adopted also as the standard for the design of new bridges on the road until the New York and New England Railroad Company was absorbed by the New York, New Haven, and Hartford Railroad Company.

It is interesting to note, however, and it is considered to be worthy of serious thought in connection with the model loading which Mr. Steinman has suggested, that although the heaviest locomotives may not run on the minor branch lines, there may be short trains of heavy cars hauled by a lighter locomotive on those lines. Instead of increasing, as was suggested, from the M-60 to the M-75 or the M-90 loading, it might be advisable to take cognizance of the fact that, on the branch lines, light locomotives may haul trains

made up of cars as heavy as those on the main lines with their longer trains and heavier locomotives. This matter should receive careful consideration.

PHILIP GEORGE LANG, JR.,* M. AM. SOC. C. E. (by letter).—For more than a generation the Cooper loadings have represented to American engineers the accepted medium for determining and recording the carrying capacities of bridges. A careful perusal of Mr. Steinman's paper indicates that it is primarily an attack on the Cooper system of loading. Certain particularly heavy engines in actual use have been selected, and an attempt has been made to develop a loading to represent the composite of these engines. The actual engines selected are extreme, and are used in special localities for special purposes.

Why such a special engine loading as M-60 should be followed by a train load of 6 000 lb. per lin. ft., is not apparent. As special engines were selected for study and comparison, there is no reason why similar cars should not likewise have been selected. Cars are in use on American railroads, that average 8 300 lb. per lin. ft. of track, and others have been proposed of more than 10 500 lb. per lin. ft. The Cooper loading provides for an increase in the following train load and thus tends to bring the average train load up to that of the engine load. The M-60 loading perpetuates a material difference in the effect of the engine and the train load on bridge structures.

The author states that "since the days of the E-40 loading, the axle concentrations have increased 100%, whereas the weight of train per linear foot has increased barely 50 per cent."

It would seem that such a statement is open to question. Although, perhaps, the engine axle concentrations have increased 100%, the weight of the train load per linear foot has increased about in the same proportion; certainly, 30 years ago, cars having capacities of 20 and 30 tons were common; to-day, cars with capacities of 55 and 70 tons are in general use, and many cars with capacities of 100, 110, and 120 tons are in operation and proposed for general use.

Again, the author states that "it [the new loading] should be from 25 to 50% heavier than the average present loading." Just what is meant by this statement is not readily discerned. The engines investigated by the author have an effect on bridge structures approximately equivalent to Cooper's E-73, which is equivalent to the M-60 loading, with the exception that the author's loading provides for a train load of 6 000 lb. per lin. ft., whereas, Cooper loading provides for a train load of 7 300 lb. per lin. ft., which more nearly approximates present-day equipment and rolling stock.

The simplified loading and the load formula are essentially an M-60 loading. Each of these forms is merely a return to the old and, perhaps, original idea of simplified loading to be used in order to make for ease in calculation. Such a loading has no basis in fact, and is simply prepared with the idea of making it produce stresses in bridge structures approximately equivalent to the motive power and rolling stock in use, or, in this case, approximately equivalent to the proposed typical engine designated as "M-60". There does not seem to be

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any valid reason for the use of such a so-called "simplified" form of loading, and when a typical loading for the design and re-calculation or the rating of bridges in service is determined, a complete set of moment, shear, and floor-beam reaction tables may be prepared. Such tables for the Cooper loadings have been in use for many years and simplify the calculations; certainly, as much so as the proposed simplified loading, or the load formula.

It will be shown that the loading proposed by Mr. Steinman does not vary appreciably from the Cooper system of loading, and that the Cooper system represents the actual loads in service more nearly than the proposed M-60 loading.

In Table 10 is given the moments and shears for span lengths varying from 10 to 300 ft. The equivalent uniform load is shown for the E-60 and the M-60 loadings and also the actual E-rating for M-60 and the E-rating for M-60 as given by Mr. Steinman. The E-ratings given in the paper differ from those computed by the writer. For moments, with the span lengths shown, this difference ranges from 0.6% to 14.2%, with an average of 3.1%, and for shears from 2.4 to 7.4% for span lengths varying from 10 to 80 ft., with an average of 5.25 per cent. It would seem that this constitutes a rather remarkable variation where an effort is made to supersede the existing Cooper loadings.

TABLE 10.—Comparative Table—Moments and Shears.

Moments						Shears					
Span, in feet.	E-60, equivalent uniform load.	M-60, equivalent uniform load.	E-rating for M-60.	E-rating shown in Table 2.	Percentage of variation.	E-60, equivalent uniform load.	M-60, equivalent uniform load.	E-rating for M-60.	E-rating shown in Table 2.	Percentage of variation.	Span, in feet.
10	7 500	7 880	63.0	72	14.2	9 750	10 880	67.0	72.0	7.4	10
20	6 190	7 125	69.1	73	5.6	7 590	8 815	70.5	74.0	5.0	20
30	5 475	7 000	76.7	74	3.8	6 300	7 915	75.5	73.0	3.3	30
40	4 920	6 340	77.3	75	3.0	5 655	7 120	75.6	72.0	5.0	40
50	4 565	5 715	75.2	73	3.0	5 235	6 480	74.3	70.0	6.1	50
60	4 330	5 335	73.9	72	2.6	4 900	6 190	75.8	71.0	6.8	60
70	4 180	5 100	73.1	72	1.6	4 740	5 940	75.2	71.0	6.0	70
80	4 050	5 000	74.1	73	1.5	4 660	5 650	72.7	71.0	2.4	80
90	3 950	4 935	75.0	73	2.7	4 565	5 335	70.0	70.0	....	90
100	3 865	4 830	75.0	73	2.7	4 500	5 160	68.9	69.0	....	100
120	3 845	4 615	72.1	71	1.7	4 340	4 920	68.0	68.0	....	120
140	3 790	4 420	70.0	69	1.4	4 200	4 729	67.4	67.0	....	140
160	3 730	4 200	67.6	68	0.6	4 100	4 555	66.7	67.0	....	160
180	3 640	4 025	66.3	67	1.0	4 000	4 415	66.2	66.0	....	180
200	3 555	3 835	65.2	66	1.2	3 920	4 300	65.8	66.0	....	200
250	3 370	3 545	63.1	65	3.0	3 765	4 075	64.9	65 0	....	250
300	3 255	3 310	61.0	63	3.2	3 640	3 915	64.5	65.0	....	300

Note.—E-60 = two of Cooper's E-60 engines, followed by 6 000 lb. per ft., or two 75 000-lb. axle loads, 7 ft., center to center.

The statement has been made that "the proposed loading has much to recommend it, particularly as applied to design. It more nearly represents future developments in train load to be expected in this country, and insures a more uniform design of bridges."

It is doubtful whether such is the fact. The M-60 loading approximates the heaviest engines now in use, but is followed by a train load of 6 000 lb. per lin. ft. of track, which does not represent present-day maximum train loads.

There is, therefore, too great a disparity between the engine and the following load in the case of the M-60 loading. This disparity does not exist with the Cooper loading, particularly when M-60 and the present loadings are compared with Cooper's E-73.

Fig. 17 shows moments and shears for existing heavy engines and the three forms of the author's proposed loading in comparison with Cooper's E-73. The stress-producing effects of the heavy locomotives approach Cooper's E-73 for the longer spans if the following train load is increased. It may also be noted that the actual engines are represented more nearly by the Cooper E-73 than by the M-60 loading. For moments, the variation of the actual heavy locomotives from E-73 is from 4 to 13%, whereas, for M-60, the variation is from 6 to 16 per cent. For shears, the variation of the actual engines is from 1 to 12%, whereas, for the M-60, it is from 4 to 12 per cent.

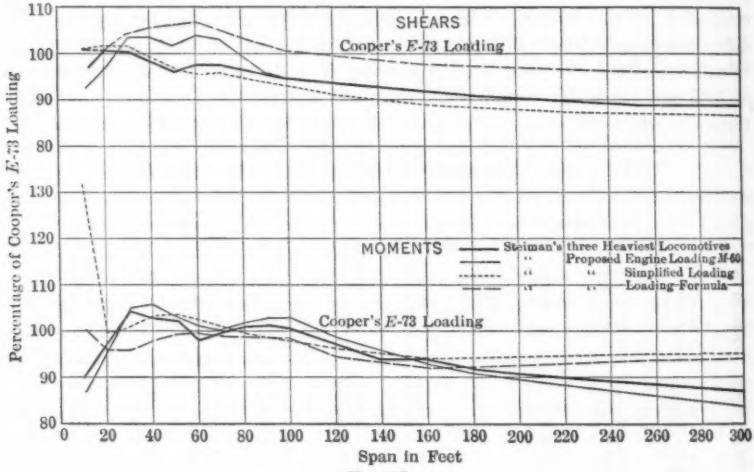


FIG. 17.

The further statement has also been made:

"In view of the present universal use of the Cooper loading, it may well be continued for the classification of old structures, in spite of some of the shortcomings pointed out by Mr. Steinman, but there is clearly need for something other than the short Consolidation locomotives of the Cooper series when it comes to proportioning the parts of a new bridge."

Evidently, it is proposed to use one method or system for the design of new structures, and to continue in use a different system for the rating of existing structures and for the determination of their carrying capacity. This would require a structure to be re-calculated for the other system. The bridges now in service cannot be re-calculated for a new system of loading. The work involved is too great, and engineers in charge of railroad bridges have not the time to change the records. Statistics indicate that there are about 250 000 bridges in use on American railroads. Presumably, complete records and calculations for these bridges exist.

In a study of the length of bridge spans, more than 5 000 railroad bridges have been classified, this study being representative of general bridge conditions in the United States. Exclusive of timber trestles, it was found that these bridges might be classified as to spans, as follows:

Less than 120 ft.	96%
From 120 to 149 ft.	1½%
" 150 to 199 ft.	1%
" 200 to 249 ft.	1%
" 250 to 300 ft.	4 spans
More than 300 ft.	11 spans.

It is apparent from the foregoing tabulation that the plate girder is the most prevalent type of bridge. Spans in excess of 120 ft. are relatively few, and, undoubtedly, the few bridges that exceed 300 ft. in length, when designed and built, are the subject of special consideration. The 1920 A. R. E. A. Specifications for Steel Railway Bridges treats of spans up to 300 ft. in length. In view of the classification of bridges mentioned previously, this discussion has been limited to the consideration of spans of less than 300 ft. Fig. 17 shows that for spans from 10 to 120 ft., there is practically no variation between the Cooper series of loadings, the engines in use, and the author's proposed loadings—certainly not sufficient variation to justify the adoption of a new system of loading.

In this connection, not only structures to be designed in the future must be considered, but also the vast number of existing structures in use, for which elaborate records are maintained in order that motive power may be assigned so as to secure the maximum benefits, considering the ultimate carrying capacity of the structures.

The author's study is appreciated, particularly as it has the effect of drawing attention to the question of the load to be used in the design of railroad bridges. Except for the advantage to be derived from such discussion, it is not felt that a convincing argument for discarding the Cooper system has been developed.

H. T. Welty,* M. Am. Soc. C. E. (by letter).—The author is to be complimented for his valuable contribution to this subject, which, in the writer's opinion, constitutes a strong argument in favor of retaining the Cooper system of loading, with possibly a modification in the uniform load following the series of wheel loads.

Many diagrams are shown, which will be of great value for reference, but to the writer, the most illuminating data are those given in Table 2 showing equivalent Cooper ratings for various spans of the proposed composite loading, M-60. The majority of railroad bridges have span lengths of less than 100 ft. It will be seen by referring to Table 2 that the moments produced by the M-60 loading for such spans are equivalent to Cooper's E-73 for spans of 20, 50, 80, 90, and 100 ft., and vary, for the others, from E-73 by less than 3 per cent.

Even for spans up to 160 ft., the variation is only 7%, and this is because Mr. Steinman uses a uniform load of 6 000 lb. per lin. ft. for his M-60 loading,

* Engr. of Structures, N. Y. C. R. R., New York City.

whereas, the Cooper E-73 would carry with it a uniform load of 7 300 lb. per lin. ft. Should the Cooper loading be modified to the extent of using 6 000 lb., which would be necessary to obtain a correct comparison with Mr. Steinman's M-60 loading, the agreement between the two would be still more striking, as this would raise the equivalent E-ratings for the longer span lengths. Also, for shear, the agreement is nearly as close, the Cooper loading giving slightly higher shear stresses.

The author takes a 140-ft. span as an example to show the deficiencies of the Cooper loading. He shows that, in the members of such a span, the Cooper loading would give stresses varying by as much as 37% from those produced by the N-1-S engines of the Pennsylvania Railroad Company. Such variations will always be found, some greater, some less, where actual locomotives are compared with a conventional set of wheel loadings, whether such loadings are the Cooper or the proposed M-60 loading.

The Cooper E-loading was adopted for the A. R. E. A. 1920 Specifications for Steel Railway Bridges, after thorough discussion and consideration. Some of the reasons in favor of the retention of this loading have been given in a paper* by Mr. W. S. Bouton and the writer.

The writer can see nothing of practical value to be gained and much to be lost by changing from the Cooper to the proposed M-60 loading, or to any other loading yet devised.

CLEMENT C. WILLIAMS,† M. AM. SOC. C. E. (by letter.)—This able paper is a valuable contribution to the science of structural design and should promote economy, adequacy, and consistency in the design of railway bridges. Although the Cooper loadings were a boon to structural engineering in one sense, they have been a detriment in another, inasmuch as they have caused the Profession to be so satisfied with a good device as not to seek a better one.

The writer in classifying many existing railroad bridges according to the Cooper loadings has observed the inconsistency of design required by that system. Mr. Steinman's statement that an error of 20 to 37% results from expressing modern loadings in terms of the Cooper engines does not indicate the seriousness of the lack of balance in most designs, for to find a structure the chords of which would class as E-50 and the web or floor of which would class as about E-25 is common, notwithstanding that the structure is fairly satisfactory as to actual requirements.

The Consolidation type of locomotive will probably never again be the prevailing type and, therefore, it represents, in its close concentration of load, a system of loadings different from those to which railway bridges will be subjected. Hence, there seems to be little excuse for continuing the inconsistent designs which result from this antiquated conventional loading. In fact, as long as it is continued, it will be common practice to remove old bridges because some of the main members and details are inadequate, while much of the structure is amply strong to carry the heavier loads to which it is subjected.

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* Am. Ry. Eng. Assoc. *Bulletin No. 219*, September, 1919.

† Prof. of Civ. Eng. and Head of Dept., Univ. of Illinois, Urbana, Ill.

The Cooper system of loading has demonstrated the advantages of a conventional system of wheel loads as a basis of design for railway bridges, and probably the present need is a revision of the system rather than a radical departure in the type of loading.

The results obtained by the Special Committee on Stresses in Railroad Track reveal the effect of wheel concentrations in a new light and seem to call for accentuating this effect rather than minifying its importance by the use of equivalent uniform loads. Unless locomotive equalizers can be improved, it might be desirable to break the uniformity of axle loads and ascribe additional load to the main driving axle. In the writer's opinion, economy of design will be hindered rather than enhanced by the adoption of such an equivalent uniform load as that which the author proposes.

It is doubtful, however, whether a single system of wheel concentrations can be adopted, which will meet the varying needs. Many cantilever structures for carrying locomotives must be designed, for which close concentrations of wheel loads lead to grossly inaccurate results; therefore, the needs seem to require several different types of wheel concentrations, to be applied according to the anticipated development of locomotives as demanded by the topography and traffic conditions of the contiguous districts. One of these types certainly should be similar to that represented by the author's M-series, and the other extreme should be one of about two or three concentrations, as represented by the Atlantic type of locomotive, to be used especially for short spans, whereas, the third might be an intermediate type similar to the Cooper engines. The development of locomotives in recent years has shown the Mallet locomotive as the logical type for heavy loads and steep grades, hence, the design of structures on such roads should be predicated on a locomotive loading of this type. The demands of topography and of traffic in so far as they affect locomotive types can be predicted with a fair degree of reliability and bridges should be designed accordingly.

Attention should be called to the desirability of considering the entire problem of loads and stresses together rather than separately. The question of loadings to be used is not more than half the case, for many of the present and past inconsistent or unbalanced designs are due to illogical selections of unit stresses quite as much as to illogical schemes of loading. Every one knows that 16 000 lb. per sq. in. does not represent the limiting safe working stress in steel in direct tension. There is little uncertainty about the strength of materials; the uncertainty lies in the loads to which the structure is subjected. The logic of the situation, therefore, would demand that the factor of safety be inserted in the loadings rather than in the unit stresses. This arrangement would make a consistent design as between dead and live load, which is not obtainable under most existing specifications. Indeed, this illogical situation permeates nearly all structural steel design—buildings as well as bridges. Specifications governing reinforced concrete design are decidedly in advance of those for steel in this respect.

Mr. Steinman has opened an important question and has made a valuable contribution, but the writer would urge that the entire question of loads and

stresses be considered together, because a simple change from the E-series to an M-series is not likely to afford a permanent improvement. When a custom or device has become so ingrained in the education of engineers through years of use as the Cooper loading, considerable professional inertia is to be expected when an improvement is proposed. It is to be hoped, however, that the author's valuable contribution will lead to the introduction of a system of loadings and stresses that will yield a more consistent and economical design for railway bridges.

JONATHAN JONES,* M. AM. SOC. C. E. (by letter).—The following assumptions underlie the author's method:

(1).—On any important railroad, the structures should be designed for the typical rolling stock of that or any other railroad, with a consistent factor of safety, rather than for the typical rolling stock of the home road, with a consistent factor of safety, but subject to certain over-stresses from rolling stock of other types.

(2).—As locomotive weights increase hereafter, they are likely to increase by the retention of present types and wheel spacings, with a proportionate increase of each wheel load.

(3).—There is an advantage to be derived from having a uniform loading specification throughout the country.

If these assumptions hold, the author's work is so searching and instructive that it becomes authoritative, and his recommendations deserve to be put into force.

These were Cooper's assumptions also, and that they have not generally held is indicated by the number of railroads which use something other than the Cooper loadings, also by the discrepancies, pointed out by the author, between the heavy Cooper system and present-day developments.

Are the assumptions of to-day, then, better than former ones? The writer hopes that they will be thoroughly discussed by mechanical engineers; certainly, the whole matter must be referred to the American Railway Engineering Association before these assumptions can be disposed of.

Regarding Assumption (1), if the only negative argument was the present-day distinctions of railroad ownership, management, and design, one could foresee these distinctions breaking down, and rolling stock traveling promiscuously as it did during the World War. However, in locomotive types are there not districts, and, therefore, groups of railroads, which always will be well served by one type and poorly by another; so that if extra metal was placed in a certain truss member to accommodate this latter type at the normal unit stress, money would have been spent, that might well have been saved.

Regarding Assumption (2), will the development of electric traction introduce a type so different as to re-determine some of the author's maximums? Or, if the present types remain as the normal types, will they grow by mere proportionate increase in weight, or must not the wheel-bases increase? Will weight per foot of train increase in proportion to locomotive weights? It

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seems to the writer that before any given railroad, well satisfied with the correspondence between its present designing stresses and the author's M-50, should specify M-50 plus an allowance for 50% overload, it might properly assume its present locomotive type, expanded both in weight and in dimensions, and compare the future stresses, thus calculated, with M-75 before accepting the latter.

With Assumption (3), the writer is in accord. Each step in bridge design from the preliminary estimate to the finished details would be simplified, cheapened, and increased in accuracy through the building up of standards of comparison, if more railroads would yield their special loadings and agree to a common standard. Therefore, the writer hopes that, even though the permanency of a new standard adopted to-day, may be voted dubious, one may be adopted, if only as a second step toward the ultimate—provided there is a pledge of adoption by sufficient additional railroads and an assurance of its persisting for, say, two decades more.

For the purposes of such a transition, a wheel-load diagram would be the least radical and the most acceptable. It is regrettable that the proposed M-type differs from any existing type in the change of driver spacing and driver weight in the two groups. This will make it difficult to "sell" the type to literal minded executives whose reactions will be that it is like neither their own engines nor those of any other railroad, and, therefore, cannot be reasonable. The logical answer to this is found in the author's derivation of the type. In addition to the advantages listed by the author, a diagram has the advantage, as a transition step, that it is a picture which invites review as mechanical developments come. The excess concentrations, or the formula proposed, if once adopted, lose all significance, except that they get a certain mathematical result. They are stopping, not starting, points in the reasoning, and until engineers are agreed that the starting point is beyond further revision, they should not bury it in a formula, but keep it out where everybody can look at it. This latter refers to the choice for publishing in a specification. The designer, however, would probably forget the wheel-load picture, paste Fig. 2 or Plate X in his handbook, and perhaps memorize Formula (4) for emergencies, feeling more than a little grateful to the author for precious hours of computation saved him thereafter.

GLENN B. WOODRUFF,* M. AM. SOC. C. E. (by letter).—The author deserves the thanks of the Profession for the study he has given to this subject.

The Cooper system of loadings does not fit present-day conditions, and this divergence is likely to be increased by future developments in engine and train loadings. However, the Cooper system is in general use, and, before making a change, it would be well to take all possible measures in order that the changed loading will meet the developments of the next thirty years as well as the Cooper loading has done in the past. A thorough study of present and probable future developments of motive power and equipment design should be made by the proper committees of the Society and of the American Railway Engineering Association.

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In considering the engines selected by the author as a basis for his "composite loading", the following suggestions are offered:

1.—One of the locomotives shown was built for pusher service and moves only over a limited part of the owning railroad. Others were built for roads having peculiar operating conditions. It is a question whether such locomotives are likely to come into general use.

2.—Engine No. 3 is probably somewhat ahead of its time, considering the present trend of locomotive loadings. The writer believes that the "composite loading" does not provide for short spans as it should and suggests adding a hypothetical engine of the Pacific type with axle loads of about 84 000 lb. on drivers. This would necessitate modifying the M-loading by specifying an alternate loading that would govern for short spans.

3.—Although the uniform load of 6 000 lb. may represent present-day conditions, the writer believes that it is not enough to be in keeping with the locomotives shown. There is probably more chance for a large increase in the train load than in the engine load. The writer would suggest an increase of about 10% in the train load of the proposed M-loading.

Considering the three alternate solutions proposed by the author, the writer prefers the continued use of the wheel loads. The saving of labor by the "simplified" methods is slight. By the use of tables and diagrams, the calculation of live load stresses is an easy task. It is a simple matter to prepare influence lines for any structure and then use the values given on Plate X. Having a set of concentrated loads in mind, one is less likely to err when it is necessary to give a hurried ruling on the acceptance or refusal of a heavy shipment.

The divergence between the "simplified" loadings and the "composite loadings" is nearly as great as between the composite loadings and the Cooper E-65. The writer fails to see any valid argument in favor of their use. Instead of designing for the M-60 loading, with a provision for 25% overload, the writer suggests designing directly for M-75, both for simplicity and economy. For economy of metal, it is desirable that all members should reach their limit under the same load, and the logical method of securing this result is by designing for ultimate load.

T. Kennard Thomson,* M. Am. Soc. C. E. (by letter).—The author deserves the thanks of the Society and the Profession for his valuable contribution on engine loadings. The writer recalls the unnecessary labor due to the great variation of loads, not only as specified by different roads, but also as adopted by individual roads. When he began working for a bridge company about thirty-six years ago, the entire design of bridges was in the transition period. Not only did each railroad have its own engine diagrams, but it also had its own peculiarity of design. Designs were so different that a glance would enable one to tell the railroad for which a shop plan was being made. The same applied to the bridge companies, as one would know, in most cases, on seeing a completed bridge, what company had built it.

* Cons. Engr., New York City.

In the intervening years, however, the railroads and the bridge companies have adopted the better and have discarded the poorer features, so that there is now much more uniformity. Probably no single feature has changed more than the engine and train loads. The efforts of each railroad to design its bridges as close as possible to one or two types of engines, has been, in many cases, false economy.

If Mr. Steinman's valuable paper causes the adoption of a few standard engine load diagrams, much mechanical labor in estimating will be saved, and the result will probably be fewer errors, more uniformity of design, and much less likelihood of frequent replacement.

As engineers, other officials, and even owners of railroads are changed so often, and as the same freight transfers from one road to another, it seems absurd to have such an infinite variety of engine loads adopted, only to be superseded by others in a few years.

The late Theodore Cooper, M. Am. Soc. C. E., was probably the first to devise an engine diagram which was generally adopted. The highway bridge companies often simplified matters by specifying a uniform load, plus a concentrated load, which saved much labor.

The writer hopes that Mr. Steinman's paper will result in the adoption of a few standard systems of loading, which will be tabulated for a uniform load per linear foot, and that these loadings will include long-span bridges for highway, combined highway, and railway purposes.

C. R. Young,* Assoc. M. Am. Soc. C. E. (by letter).—The author is to be congratulated for the service he has rendered the Engineering Profession in placing before it, data so convincing as to leave no other course open than to revise the present obsolete locomotive loadings. Much discussion has been given, in recent years, to the alleged inapplicability of the Cooper system of conventional loadings, but in most cases this has been of a desultory and general character. The author has established that, for heavy loadings on first-class railway systems, the Cooper system is inadequate and that the new proposed system would give safe and logical results. It now remains for the railway systems to make the next move. It is to be hoped that this will not be long delayed, and that, in particular, the American Railway Engineering Association will reconsider its decision to abide by the Cooper system.

Caution is needed in attempting to apply the author's proposed system of loading to railways carrying only moderately heavy traffic. Although his loadings are admirably suited to the heavier lines operating with steam locomotives, they are not so closely applicable to average or second-class lines on which existing locomotives, and those likely to be used for some years to come, are of a type distinctly different from those used by Mr. Steinman in the establishment of his composite loading. For this reason, it may be preferable to retain on some railways a loading system that more closely approximates the actual loadings in use than any class of the author's standard loading. The electric locomotive is becoming important and should be considered in any system that is devised to be used for a long time.

* Assoc. Prof. of Structural Eng., Univ. of Toronto, Toronto, Ont., Canada.

Of the three types of loading proposed, the writer prefers the loading formula. As each of these loadings is empirical, there seems to be no good reason to halt part way in the determination of stresses in bridge structures. Rather than adopt a loading that is conventional, it would seem best directly to adopt stresses that are conventional. There is no greater offense in the second method than in the first. It may be desirable to retain the wheel loading for investigation of special parts of structures receiving their maximum stresses under short loading, but, in most cases, the writer believes a distinct advantage would be derived from proceeding directly to the use of the loading formula.

EDWARD GODFREY,* M. AM. SOC. C. E. (by letter).—The author's able paper clearly demonstrates that, for the heaviest type of locomotive, coupled to the heaviest loads usually carried, the Cooper loading does not give consistent stresses nor economic design. This is, however, not a conclusive reason for discarding the Cooper standard of loading. If all bridges could be rebuilt, it would be desirable to establish a standard of loading to correspond with the latest in modern engine construction. Bridge specifications, however, are used largely to judge the capacity of existing bridges as well as a standard by which to build new ones. Furthermore, perhaps, a large mileage of the railroads in the United States will never be called on to carry the very heavy engine loads that are in use on a few of the roads. It would mean the rebuilding of many bridges and even the laying of heavier rails to enable these roads to carry heavy loads. As a criterion for judging the bridges on such roads or the building of new ones, these heavy loads, therefore, are just as inappropriate as the Cooper loading is on roads where the modern heavy engines are in use.

It is difficult to discard a system of calculating stresses that is as interwoven with railroad bridge design as the Cooper standard loading. By using two simple modifications of the Cooper standard of loading the author's principal objections can be overcome, and it can be made to give results as nearly equivalent to the modern heavy loading as practical considerations demand.

One modification is to use heavier axle loads in the pair of concentrations for stringers or short girder spans. The other modification is to use a train load less than that called for by the class of engine in the standard. For example, let an E-70 engine be followed by 6 000 lb. per ft. instead of 7 000 lb. per ft. By a simple rule such as this, the lack of economy in long spans is eliminated.

After a standard of design is once established, it should be adhered to with little or no departure. One can believe this and still be aware of the fact that the exact stresses computed may never be even closely approximated in the bridge. The ends of safety and stability are served, and this is the principal object. The main reason for rigid adherence to certain rules in calculating stresses is to have a definite standard of design. In competitive

* Structural Engr. (Robert W. Hunt & Co.), Pittsburgh, Pa.

design, this is particularly useful, as without a definite standard, comparisons could not be made. It is not so necessary, therefore, whether a diagram of loading agrees exactly with the axle loads of a locomotive, as long as the heavy concentrations in the diagram practically agree with those to be used, or as long, in fact, as the load per foot is almost the same.

The only bridge loading for calculating stresses that can be tolerated by designers is either a uniform load, a system of concentrations, or a combination of both. To introduce a new system of concentrated loads would place a great burden on designers who are familiar with the Cooper loading and have many tables, etc., based on it.

It would not be wise to mask bridge loading by a formula, as proposed by Mr. Steinman. The general manager of the railroad will say, "never mind the formula, what kind of engine and train loading can I run over this bridge?"

To try to interpret the strength of an old bridge, or a new one for light loads (and such will be built for many years to come), with respect to the heavy Mallet type of loading, would be just as awkward as it is to interpret the strength of a heavy bridge by the use of the Cooper loading.

The Cooper loading standard is needed and can be used for many years to come, and with slight modifications it can be made to suit any modified locomotive loading with all the accuracy that actual conditions demand.

A. F. Robinson,* M. Am. Soc. C. E. (by letter).—The author has condemned the E-loadings and submitted a new loading. In the writer's judgment, the Cooper loading is superior in every way to that proposed by the author. The simple statement in the paper that the Cooper loading is out of date does not necessarily make it so. The author has submitted nothing whatever, except this bare statement.

Before adopting the Cooper E-loading, when making up the A. R. E. A. Specifications of 1920, Committee 15 of that organization studied the question with extreme care. A sub-committee submitted a report which showed that the moments and shears for the E-series loadings compared with those for all the heavy engine loadings.

In the writer's opinion, it is not necessary to try to make up an ideal engine loading which shall consider in any way the Mallet type of locomotive, as it will not be necessary to provide for many engines of that type. Experience has demonstrated that the large Santa Fé type and the Mikado type of locomotives will give much better net returns in service than any of the Mallet types. This being the case, there is no need of providing especially for the Mallet engines. As locomotive designing progresses, it may be necessary to modify or re-construct the E-loadings, but it is not necessary for the present and the immediate future. A careful study of the report of Committee 15 of the American Railway Engineering Association, submitted with the 1920 Specifications of that organization, will clearly demonstrate what the writer has stated.

The E-loadings follow very closely the weights, moments, and shears, as given by the heavy power in service, which includes the Mallet type of engines.

* Bridge Engr. System, A. T. & S. F. Ry., Chicago, Ill.

This E-loading furnishes a unit or basis for comparison which has been generally adopted. The locomotive loading proposed by the author is in advance of anything that is likely to be used on the Atchison, Topeka and Santa Fé Railway for many years to come. An engine loading that is materially in advance of the engines that are being used at the time, should not be adopted.

Although the E-loading can be criticized in many ways, it still meets present-day requirements very well and is much to be preferred to the loading submitted by the author. The M-60 loading does not give any check on the effects of the motive power now in service. Before this system of loading is generally adopted, the locomotives must be increased materially in weight and a different kind of a road-bed, together with much heavier rails, will be necessary. The writer is in favor of adhering to the E-loadings. This is a case where the cure is much worse than the disease.

Lewis E. Moore,* Assoc. M. Am. Soc. C. E. (by letter).—Some of the most difficult questions with which railroad bridge engineers have to contend are the probable future increases in loads and whether the useful life of a bridge will be ended before its capacity is exceeded. Engineers hardly could have been expected to foresee or to provide in their bridges for the tremendous increases which have taken place, both in the total weight of locomotives and in the wheel concentrations. To predict what will happen is difficult, but that is no argument for failing to make as wise provision for the future as circumstances and conditions seem to warrant.

For the heaviest present-day loading, the Cooper series does not meet the requirements. Perhaps, few railroad companies at present use locomotives like those on which Mr. Steinman's diagrams are based, nevertheless, the tendency of loads, both total and concentrated, has been upward for a number of years. Any change in loading which better meets the tendencies of locomotive design and which can be used as a more or less general "yard-stick", is an advance. The fact that all lines do not use such enormous locomotives as the maxima treated in the paper, is no reason for condemning the system evolved in it. It is possible to use any part of Mr. Steinman's loadings in any particular case. As to whether Loading Nos. 1, 2, or 3, is used would depend largely on the ideas of the individual engineer. Having the loading in such shape that one can check the resulting stresses by judgment to a certain extent, is, perhaps, to be preferred to a single loading formula, especially as difficulties of computation cannot be allowed to stand in the way of excellence or reasonable accuracy of design.

The omission of electric locomotives from the paper is rather unfortunate, because it seems probable that their use will increase materially in the future, particularly on congested lines with heavy traffic. To see just where the electric locomotive fits among the engines used by Mr. Steinman will be of interest. It would seem that the stress-producing effects of a proposed Norfolk and Western freight locomotive† having four axle loads of 70 250 lb. each, concentrated in a distance of 16 ft. 6 in., with another similar series of loads 22 ft. away, would be somewhere between those of Loadings Nos. 2 and 3, sug-

* Cons. Engr., Boston, Mass.

† *Engineering News-Record*, September 14, 1922, p. 437.

gested by the author. Accurate computation might not confirm this somewhat superficial conclusion.

The size of the engine governs the length of the train, but it does not govern the weight of the individual car. In most cases, any locomotive, however small, that a railroad possesses, is capable of pulling a train of maximum weight per foot, which will fill its longest bridge span, so that whatever variations are made in the locomotive loading, the loading following the tender should be kept to the maximum and should be the same whatever the type or weight of the locomotive. This should be so, irrespective of the location of the railroad, as long as it has trunk-line connections, and, also, of whether the bridge is on a main line or a branch.

The exact trend of locomotive development for the future is problematical, but owing to the present tremendous wheel concentrations and the resulting wear on road-bed and track, it would seem that the limit has been almost reached. This has been said in the past, and yet wheel concentrations have increased. The limits of clearance have now been quite well filled and as the cost of increased clearances is practically prohibitive, it is perhaps more reasonable to say that the limit of individual wheel concentrations has now been reached than it would have been some years ago. When the dimensions of width and height are entirely utilized, only length remains in which expansion can take place. It would seem, therefore, that locomotives may become longer in the future, with perhaps a different wheel arrangement.

Locomotive cylinders and reciprocating parts and rods have become so heavy that development may take place along entirely different and new lines. This might mean the substitution of a vertical high-speed engine of the marine type, probably condensing and compound, geared to the axles, instead of the present enormous pair of single cylinders. Such a construction should have a number of advantages toward the reduction of weight and possible increase in fuel economy, as well as a lower cost for repairs. Such construction would make it possible to install an engine on the tender and so use for tractive purposes a dead weight which must now be hauled with no compensating advantages. Development of this kind would probably result in wheel arrangements somewhat similar to that of the existing electric locomotives. Such locomotives, if properly designed, could be run in either direction with equal facility, thus obviating the use of turn-tables and the periodical increases in turntable lengths, which have been one of the many difficulties facing railroad engineering departments. The high-speed engines of the marine type could be made interchangeable so that if repairs were necessary to the engine and not to the boiler, the engine could be removed and another substituted in its place, which would make it unnecessary to tie up the whole expensive machine for repairs to, perhaps, a minor part. A locomotive of the type outlined could be made to have a torque practically as steady as that of the electric motor. The balancing of the engines would be much simplified and the wear and tear on the track and equipment, due to lack of balance, would be practically eliminated.

Railroad motive power executives, if confronted with this development, will say that Trevithick used a vertical cylinder locomotive and discarded it because of the vertical vibration. To those who may raise this objection, the writer will say that Trevithick's engine was not balanced and will also ask them to consider the automobile.

In the desire to prophesy a little, the writer has apparently wandered far from the main subject, but perhaps not so far as may appear. Such a development would result in locomotive loadings which would more closely approach the general characteristics of those given in the paper, although they would not necessarily have such large wheel concentrations. Mr. Steinman deserves great credit for his paper. The writer believes the paper to be progressive and that the system of loadings proposed by the author will be quite generally adopted.

CHARLES B. WING,* M. AM. SOC. C. E. (by letter).—The author has performed a valuable service in showing the discrepancies existing between stresses obtained by using the Cooper E-system of engine loadings and those caused by seven types of engines now in quite general use. Calculated static stresses in railroad bridge structures subjected to train loads serve two purposes: First, to form a judgment as to the safety of structures now in use; and, second, to form a basis for the design of structures yet to be built.

Engineers charged with the responsibility of maintaining existing structures stressed by new types of locomotives of increased weight, naturally choose the wheel-load method of analysis and for this purpose use the wheel concentrations and spacings belonging to the heaviest engine operating on the division on which the structure is located. In the writer's opinion, no system of engine excess or equivalent uniform load analysis is as satisfactory for this purpose as the method of using the actual wheel loads and spacings of the heaviest engine in operation.

As a basis for designing new structures, it is advisable to look forward to probable changes in engine loadings. If, as the author contends, the present standard Cooper E-system is no longer adequate, a new standard should be adopted. The author's proposed M-system of loadings is worthy of consideration and may prove to be the needed modification of the Cooper E-system.

The determination of the static stresses in the members of bridge trusses, due to a system of moving concentrated loads, is a comparatively simple problem of analysis.† Stresses thus obtained, properly modified for impact, form a safe and satisfactory basis for bridge design.

The writer can see no advantage to be gained by the adoption of either of the substitute systems mentioned by the author. The difficulties of wheel-load analysis mentioned by many writers in the past are more imaginary than real. Given a moment diagram and a fair knowledge of stress analysis, the time consumed by either of the proposed systems of analysis is approximately the same. As the proposed substitute systems are based on the actual wheel concentrations, there seems to be no good reason for their preference.

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* Prof., Structural Eng., Stanford Univ., Palo Alto, Calif.

† "General Criterion for Position of Loads Causing Maximum Stress in Any Member of a Bridge Truss," by L. M. Hoskins, *Transactions*, Am. Soc. C. E., Vol. XLII (1899), p. 240.

R. O. STEWART,* ESQ. (by letter).—The author has chosen those few types of locomotives, which produce peak points on a moment curve, and has compiled a series of arbitrary loadings which will give approximately an enveloping moment curve for such peak points. There is nothing new or unusual about the two alternative methods of handling this arbitrary loading, and these alternatives do not require or merit special consideration, as the same principles can be applied to any conventional system of concentrated loadings.

The essential requirements of a standard bridge loading for any given railway are, as follows:

(*a*).—One standard system of conventional loading must be used for all bridge structures, such loading to be increased or decreased in fixed ratios for heavy or light structures. For purposes of design, this requirement is desirable and, for the classification of old structures, it is essential.

(*b*).—The basis of any standard bridge loading should be an average of the heaviest of the existing or probable locomotives in greatest common use on the railway, or, in other words, the standard loading should be designed to give the most economical structure possible for the class or strength of structure most generally required. The standard loading should also be such that proportional increases or reductions of it will produce the smaller number of heavier or lighter structures, without too great a sacrifice in economical design.

(*c*).—As the tendency has been (and may possibly continue to be) for the weight of rolling stock to increase slowly, the standard loading should tend to be more nearly normal to the heavier rather than to the lighter existing or probable locomotives.

On the basis of these requirements for a standard loading and considering not one railway, but the majority of American and Canadian railways (the customary types of locomotives being somewhat common to both countries), the writer would consider Mr. Steinman's proposed standard loading at fault in the following respects:

1.—Rather than being based on the heaviest of the regular standard types of locomotives in common use on the majority of average railways, the loading has been designed to cover only the unusually heavy locomotives among a few railways. These locomotives are probably operated only on restricted districts where special design provisions could readily be made for the particular heavy locomotives involved.

2.—Contrary to Mr. Steinman's contention, this loading cannot be reduced in fixed ratios, in a manner similar to the Cooper loading, to suit properly the requirements of railways operating only lighter locomotives or to suit the requirements of various intensities of loading necessary on different sections of any one road.

The author's M-60 is a fair loading for any railway operating locomotives of the Erie Railroad, Class P1, 2-8-8-8-2, the Virginian Railway, Class 2-10-10-2, and the Pennsylvania Railway, Class N-1-S, 2-10-2, the train load being 6 000 lb. per lin. ft. in each case. However, this is the one and only condition

* Asst. Engr. of Standards—Bridges, Canadian National Rys., Grand Trunk Pacific Ry., Toronto, Ont., Canada.

in which the use of this loading is fair or economical. For a railway operating only one or two of these heavy classes of locomotives, the author's loading is not correct.

Mr. Steinman states that "this loading can be increased or reduced in fixed ratios in the same manner as the Cooper system of loading". The bulk of the bridges of the Canadian National Railways are being designed to carry economically the Santa Fe T-1-c, Mikado S-1-e, Consol N-2-a, and Pacific J-7-c locomotives, and the writer thinks that these four classes of locomotives represent quite fairly the general loadings of the average Canadian and American railway. These locomotives do not fit the author's loading nearly as well as they do the Cooper loading, except possibly up to spans of 150 ft. In addition to this, the Canadian National Railways has a number of branch lines for which it is unlikely that any but the lighter locomotives will ever be required. The author's loading is at its worst for these light locomotives, whereas, the Cooper loading is at least a fair approximation, considering that the number of new bridges required for such loading is small compared with those required for the heavier locomotives.

The writer does not think that the Cooper system of loadings is perfect, or that it cannot be improved. Quite an extensive study of this matter was made in 1921, using the locomotives of the Canadian National Railways. Although several systems of loadings were devised, which were considerably more normal to the heaviest locomotives, than the Cooper system, it was decided that, at least for the present, the use of the Cooper system would give designs as economical as any other loading which could be evolved. Railways the requirements of which for bridge loading were heavier than those of the Grand Trunk Pacific Railway might find it advantageous to adopt a system more nearly normal to somewhat heavier locomotives than those to which the Cooper loading will agree, but the writer thinks that the author is altogether wrong in proposing as a general standard a loading based on a normal to the few exceptionally heavy locomotives which he has chosen as the "heaviest existing locomotives".

HENRY S. JACOBY,* M. AM. SOC. C. E. (by letter).—The author deserves much credit for the scientific manner in which he has investigated this subject, and the systematic form in which the paper is presented. The use of a composite standard loading and of the segments of the influence triangle for any truss member are especially noteworthy. The comparisons made between the results obtained for each of the seven heavy modern loadings that are selected for the composite standard and for the Cooper standard now in use, seem to answer most of the questions about their relationship, as far as static stresses are concerned.

The question may be raised, however, whether the proposed M-60 loading with its relatively lighter uniform train load will represent more closely the development of future actual live loads than the equivalent Cooper loading with its relatively heavier train load. This matter will require the composite judgment of a great number of railroad engineers in active practice.

* Prof., Bridge Eng. Emeritus, Cornell Univ., Bethlehem, Pa.

If the officers of any railroad on which light motive power is used, are in doubt whether locomotives of any of the types considered by the author are likely to be adopted on their line in the future, and who, therefore, would hesitate to accept a standard like M-40 or M-50, an independent investigation can readily be made to decide the question. The time required for such an investigation would be short in comparison with that devoted to this subject by the author. Only the heaviest locomotives of each class now in use on the given railroad need to be considered. The spans can be limited to those which include, say, 90% of the bridges now in use. A study of the results for these short spans will soon indicate whether greater or less differences among themselves in equivalent uniform loads are caused by those locomotives for any given section or truss member than by the seven locomotives selected by the author. After selecting the equivalent uniform loads from which a special composite standard for that railroad may be derived, they can be compared with those given by the author to see whether they differ materially from those obtained by scaling down the corresponding values due to the M-60 loading. It may also be observed whether the differences are larger for the web members or the chord members. For plate-girder spans, the moments and shears are due to the locomotives alone, except for the small number having the largest spans.

In making a choice of one of the three loadings proposed by the author, it is well to remember that locomotive wheel loads were introduced in the United States nearly fifty years ago, and that numerous discussions since that time have failed to displace them. It is illuminating to re-read the discussion* held at the Annual Convention of the Society on June 27, 1899. This is especially true with respect to forecasts made at that time regarding the increase of locomotive loads. Although a series of uniform loads determined by a formula may be more scientific, there are evident advantages in a series of wheel loads, which commend it to railroad officers outside the engineering department. With the aid of tables and diagrams, the computation of stresses is practically as simple in one case as in the other.

The fact that the American Railway Engineering Association concluded to retain the Cooper series of loading in its 1920 Specifications, after its Committee had carefully re-investigated the subject, makes it necessary to present exceptionally strong arguments to secure a change in the near future. The extensive records now in use on the rating of railroad bridges in terms of the Cooper series make it inadvisable to introduce a change involving so many short-span bridges, unless advantages are secured, which exceed in value that of the labor required.

CHARLES EVAN FOWLER,† M. AM. SOC. C. E.—A composite loading was first proposed before the Society in the speaker's paper entitled, "Revision of the Niagara Railway Arch Bridge."‡ The same general idea is being used by P. B. Motley, M. Am. Soc. C. E., on the Canadian Pacific Railway, by

* *Transactions*, Am. Soc. C. E., Vol. XLII (1899), p. 189.

† Cons. Engr., New York City.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1919.

J. J. Yates, M. Am. Soc. C. E., on the Central Railroad of New Jersey, and by others.

When the speaker broached this subject to the Committee of the American Railway Engineering Association in 1919, he found the members of that Committee so strongly in favor of retaining the Cooper Class E-loading that the matter was not pressed, as that Committee represented the engineers who are responsible for railway loadings. Thus, it is certain that the Association and the Engineering Institute of Canada must be consulted and first be satisfied if any proposed loading is to be substituted for the Cooper system, even though that is so badly out of date and out of time with modern bridge engineering.

The history of bridge loadings shows that changes must come about slowly. The speaker recalls the wheel-load diagrams with resultant panel loads, published about 1885, by the late Thomas H. Johnson, M. Am. Soc. C. E., of the Pennsylvania Lines Southwest; the ones most used were two 74.5-ton engines followed by 32-ton cars, or about 1 640 lb. per lin. ft.; two 92.3-ton engines with the same following load; and, in 1888, the undreamed-of load of two 107.5-ton engines followed by 4 000 lb. per lin. ft. About this time, J. E. Greiner, M. Am. Soc. C. E., devised the plan of using a slide-rule of special construction for sliding the train along panel pointers to obtain moments and shears. Soon after, it was discovered that equivalent uniform load curves or tables could be calculated and practical accuracy obtained as great as that by the laborious use of wheel loads. The engineers of the Youngstown Bridge Company constructed such curves for the locomotives of about thirty of the railways for which bridges were being built and no errors resulted. This method has largely continued to the present time, and it is little trouble to shift to uniform load curves of some type of composite loading. What shall this be? Manifestly, if standard axle spacings and loads for the different types of locomotives could be adopted, the problem would be simplified and the standard adopted would be more lasting. This would cover Class A for Mallets; Class B for the Santa Fé type; Class C for Mikados; Class D for electric locomotives; Class E for the Cooper loadings, and Class F for the final or composite loading. Some of the road officials already have taken action in this direction and toward reducing counterbalancing, and the rapid change to electric operation being made on many roads will cause still further consideration. The slow speed of freight engines makes the effect of load plus impact no greater, perhaps, than that of high-speed passenger locomotives of lesser total weight. The heaviest locomotives are only used on a limited mileage, whereas, on branch lines, much lighter locomotives are used than on the main lines. The new Michigan Central Locomotive No. 8 000 will haul a train of 120 cars, but the car weights or train load per linear foot are no heavier, and doubtless seldom reach more than 4 000 lb. per lin. ft. This is one example of the fact that the train loads should be from 500 to 1 000 lb. per lin. ft. lighter for each Cooper loading after E-50 is passed. This and various other matters of loading are dis-

cussed in the speaker's paper on the Niagara Railway Arch Bridge.* Thus, the problem of devising a final composite loading is somewhat complex, and must only be done by all those interested.

CHARLES F. LOWETH,† PRESIDENT, AM. SOC. C. E.—The speaker is in sympathy with the quite general desire to simplify and, at the same time, to make more practical the typical engine and train loadings used for proportioning railroad bridges.

When he first began his experience in designing railroad bridges, it was the custom to ascertain the heaviest locomotive and train loading in use anywhere and utilize them in proportioning the design of the proposed structure. Now and then, something was added for possible increase in weight of locomotives and train loadings. This method involved a great deal of painstaking work and resulted not infrequently in the boast that the structure was designed for the heaviest locomotive known; but often before the bridge was completed, a heavier locomotive was built, with altogether different axle spacings, and all the painstaking efforts for perfection of design to meet actual conditions were defeated. The author's suggestion that railroad bridges should be proportioned for a typical uniform loading would be ideal, if one could be determined that would approximately fit actual conditions and would be generally accepted and used. Such a practice would be along the lines of that of the noted bridge designer, the late George S. Morison, Past-President, Am. Soc. C. E. His method was to take a uniform train load, headed by a largely increased loading for a length of one or, perhaps, two panels, in order to allow for the additional weight of the locomotive over that of the train. The application of this loading was simple and had many advantages, but it fell into disuse, because it did not approximate closely enough actual conditions, and, perhaps, equally on account of the fact, that the loadings then generally used did not provide sufficiently for the increasing weight of locomotives and cars.

The speaker remembers the frequent arguments in those early years as to whether locomotives would continue to be built materially heavier than the largest in use at the time. Each new locomotive seemed to be about the maximum that could be designed and would be practicable; yet, as the years passed, there was a continuing succession of heavier locomotives, and the end is not yet in sight. With heavier rails and better track construction throughout, stronger equipment, mechanical stokers, and other locomotive improvements, the increasing cost of transportation, and the necessity for the utmost economy in railroad operation, it is quite possible that locomotive and train loadings may continue to increase in weight.

However, railroad transportation is undergoing an evolution and the trend in the future may not be in the same directions as that in the past. The speaker thinks that it may be within the bounds of possibility that future transportation requirements may change, in some localities at least, to a demand for smaller train units, more quickly moved. This suggestion may

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), pp. 1925 to 1930, and p. 2020 *et seq.*

† Chf. Engr., C. M. & St. P. Ry., Chicago, Ill.

smack of heresy, because until the present time, it seems to have been a generally accepted rule that economical railroad transportation requires train units as large as can possibly be handled.

Another possibility to be taken into consideration is that of the electrification of steam railroads. As the financial problem involved in electrification is so great, the speaker does not expect to see the change from steam to electric power, for general railroad purposes, come about rapidly. There will be, however, a tendency in that direction. The electric locomotives are generally lighter in weight, at least when measured by units of length, and, what is perhaps even more important, they produce less impact effect on track and structures. They undoubtedly will be less frequently double-headed for heavy trains than steam locomotives. These considerations tend to reduce the justifiable weight of train loadings.

It would be desirable if locomotive loadings could be safely standardized without getting too far afield from the actual conditions to which the structure would be subjected. A steel bridge superstructure, for the most part, is permanently located. There are parts of railroads where the traffic requirements will never call for locomotives and train loadings as heavy as those now in general use. There are other parts where the traffic requirements will call for increasingly heavier train loadings. The speaker thinks that advantage of this difference in conditions is not as generally taken as it should be. He also questions whether sufficient consideration has been given the fact that the heaviest locomotives are those used in freight service which, from their design, as well as from their use, are limited to speeds much less than those of the lighter passenger locomotives. There are also other conditions which affect the speed of trains over any structure. Speed is a factor in the impact effect of the load, and to apply the impact for maximum speeds, irrespective of the structure and type of locomotive used, is not a rational method of design.

Relative to the effect of the electric locomotive on bridges on the Pacific Coast Extension of the Chicago, Milwaukee, and St. Paul Railroad, the speaker would explain that those bridges were designed before the decision as to electrification had been fully made. The bridges were proportioned for what is known as the Cooper E-55 loading. The electric locomotives in use, although among the heaviest, if not the heaviest, which have been built for any service, are well within the limits of the E-55 loading for which the bridges were designed. The electric locomotives have practically no unbalanced and reciprocating parts so that their impact effect on track and structures, as compared with steam locomotives, is small.

RALPH MODJESKI,* M. AM. SOC. C. E. (by letter).—There is no doubt, and it has been well brought out by the author, that the Cooper standard loading does not properly represent modern loading conditions. It would be impossible to construct a rating diagram which would correspond to the power as well as to the equipment of every railroad in the United States. It has long been recognized that the Cooper diagram cannot be strictly applied to all loadings,

* Cons. Engr., Chicago, Ill.

principally because, in individual cases, the maximum wheel concentrations have no fixed relation to the train loads. Therefore, instead of calculating a bridge for the Cooper standard loading, some combined loading would be taken, as, for instance, the Cooper E-60 locomotive followed by a train load of 5 000 lb. per ft. of track. A combined loading of this kind may be fitted quite closely to the motive power and to the train loads of any railway system. It has the advantage of allowing comparison between bridges constructed at different periods; their relative rating may then be determined without extra calculations and may be easily understood by the railroad executives.

If a new loading is adopted at this time, the comparison of the rating of existing bridges that were calculated on the basis of the Cooper loading, with the new bridges calculated by such loading, would be obliterated and could only be made by the use of tables or calculations, or both.

Referring to Table 2, it may be noted that the ratings for the center moments and end shears are quite consistent for spans up to about the length of two Cooper locomotives. Beyond that length the rating decreases, owing to the increasing influence of the uniform train load. For a 1 000-ft. span, the ratings are practically equal to the uniform train load of the Cooper E-60 loading as well as to that of the proposed M-60 loading, because the influence of the locomotive load becomes small. If, therefore, all spans from 10 ft. to 1 000 ft., were calculated for a load consisting of the Cooper E-73 locomotives followed by 6 000 lb. per ft. (calling such a load E-73-6 000 for convenience), the ratings of Table 2 would become uniform for all practical purposes and would closely correspond to the author's M-60 loading. Some slight discrepancies or even inconsistencies might occur, but this would also be the case with any other system of wheel loading. Considering that unit stresses are based only on good judgment and, further, that allowances must be made for impact, unbalanced wheels, wind, tractive forces, and secondary stresses, the difference between some such loading as E-73-6 000 and M-60, will be small indeed.

If a revision of loading is advisable, would it not be better to adopt at once different classes of loading based on a uniform load and one or more "floating" concentrations, such as those proposed by the author as a second alternative, rather than resort to another system of wheel loading which may again have to be revised in the near future. The writer believes that a uniform loading with one floating concentration would fill all requirements. Some large bridges in the United States have been calculated in such a manner.

C. A. P. Turner,* M. Am. Soc. C. E. (by letter).—As a railroad train passes over a bridge structure, a certain element of time is required to overcome the inertia of the mass in producing the greatest deflections of the parts and of the structure as a whole. The greatest elongation under tension, or deformation under compression, is a thermo-dynamic function such that the mechanical energy represented by the transfer of the kinetic heat energy of molecular vibration in the resisting metal to the potential energy of deformation of strain is double the external mechanical work of the load when the stress is tension, and the reverse when it is compression. This excess energy

* Cons. Engr., Minneapolis, Minn.

is radiated to the surrounding medium or the absorption of heat energy from it takes place. Accordingly, there is an instantaneous or kinetic deformation and a static deformation which involves the thermal effect.

The relative amount of energy involved, as shown by Kelvin's thermal dynamic theory, is approximately double that of the work of the external applied forces. The full development of the static deformation requires an element of time, which in connection with overcoming the inertia of the mass, may be approximately estimated at 1 sec.

If the train is traveling nearly 60 miles per hour, its speed may be taken at 100 ft. per sec., and the vertical impact on a member would approximate,

$$S\left(\frac{100}{A+100}\right)$$

in which,

S = live load stress; and

A = length of track loaded, for maximum effect, when the initial dead load stress equals zero.

However, if the member is under dead load stress, the approximate vertical impact strain would equal,

$$S\times\left(\frac{L}{D+L}\times\frac{100}{A+100}\right).$$

The rolling load tends to set up structural vibration laterally as well as vertically, and the total impact is the resultant of the vertical and horizontal effects combined. In the ordinary pin-connected bridge, with a train speed of about 60 miles or more per hour, the lateral impact may be assumed to be approximately equal to the vertical impact effect. Therefore, for the pin-connected railroad bridge, the formula for total impact is:

$$I=S\left(\frac{L}{D+L}\times\frac{140}{A+100}\right).$$

The stiffness of the riveted structure would reduce the necessary impact allowance to about

$$I'=S\left(\frac{L}{D+L}\times\frac{130}{A+100}\right).$$

The vertical effect of impact would vary approximately as the speed of the train, whereas, the lateral effect would vary roughly as the square of that speed. Accordingly, if the rational impact allowance of the high-speed passenger train is to be compared to that of the ordinary freight train, which travels about half as fast and the lateral impact of which would be one-fourth as great, these formulas would immediately reduce, for the pin-connected bridge, to

$$I=S\left(\frac{L}{D+L}\times\frac{55}{A+50}\right)$$

and corresponding values for the riveted bridge.

It accordingly appears that the value of the passenger train impact for a Cooper E-60 loading would be nearly equal to an E-75 loading for freight train speeds. As the excessive Mallet engines are not used for fast passenger traffic,

there is little call for a radical change in the present typical engine loading when coupled with a rational impact formula. The present standard furnishes a uniform method of comparing the capacity of bridges, and tables for its convenient application are in general use.

The fact that the Cooper loading approaches as closely to the wide range of engine loading as any that can be devised and, if used in conjunction with a rational impact formula, meets all requirements, argues against introducing other and different types of typical engine loadings, which would only add confusion to present methods of investigation. If the impact formulas are considered for range of stress from tension to compression, the first member in the foregoing suggested formula would become unity and may be disregarded.

Mr. Steinman's painstaking investigation is a distinct addition to the literature of the subject, but his showing when considered in connection with rational allowances for impact, does not indicate the need of any radical change in the well established Cooper loading.

D. B. STEINMAN,* M. AM. SOC. C. E. (by letter).—The writer gave his best efforts to the preparation of this paper and was agreeably surprised by its generous reception. The interest shown and the many expressions of appreciation have more than rewarded him for his work.

The paper was intended to be suggestive rather than final, to evoke discussion, to outline the questions at issue, and to indicate methods of analysis, solution, and application. The proposed loading specification, in its three different forms, was merely tentative, and was submitted to the members of the Society as a basis for discussion or modification. It is, therefore, most gratifying to the writer to see his labors rewarded by the endorsement of the Special Committee on Specifications for Bridge Design and Construction, of the Society, in its recommended specifications for steel railway bridges.†

The three forms of loading specification proposed in the paper may be designated as follows:

M-Loading No. 1—Typical engine diagram.
M-Loading No. 2—Simplified loading diagram.
M-Loading No. 3—Equivalent uniform loading formula.

The discussions disclosed varying preferences between these alternative loading specifications. The preponderance of choice appears to have been for M-Loading No. 1, the typical engine diagram. This also proved to be the choice of the Special Committee on Specifications for Bridge Design and Construction, which adopted M-Loading No. 1 (with a slight modification) for inclusion in the recommended specifications.‡

The modification referred to was the change of the spacing of the heavy driver axles from 5½ ft. to 5 ft., so that all wheel spacings are multiples of 5 ft., which makes a more convenient and simpler loading diagram. Although not as correct theoretically as the original 5½-ft. spacing, it is practically justified by various considerations; it departs somewhat (on the side of

* Cons. Engr., New York City.
† See p. 472.
‡ See p. 477.

safety) from the composite loading at the shorter spans, yielding a margin of stress-producing effect where it is most likely to be needed under future intensification of axle loads. The Cooper loading and the modified M-10 loading are shown on Figs. 18 and 19, respectively. The greater simplicity and convenience of the M-loading, as thus graphically presented, should require no further discussion.

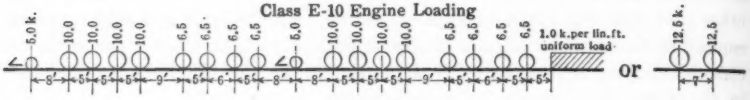


FIG. 18.

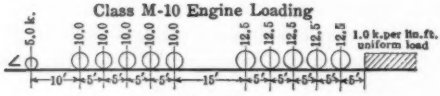


FIG. 19.

A wheel-load moment table for this M-loading is presented in Table 18,* of the Final Report on Specifications for Steel Railway Bridge Superstructure of the Special Committee on Specifications for Bridge Design and Construction. The corresponding table for the Cooper loading (Table 11† of the same report), does not compare with Table 18 in brevity and simplicity.

A diagram of equivalent uniform loads for the M-loading is presented in Fig. 20. A comparison of this diagram with Fig. 2 (which it supersedes) shows that the change from a 5½-ft. to a 5-ft. spacing has increased the stresses at the shorter spans about 5 per cent. A reference to Fig. 4, or Fig. 6, will show that this change is desirable, inasmuch as the previous M-loading was slightly deficient at those span lengths.

A chart of equivalent uniform loads for this M-loading is presented in Plate XIII (superseding Plate X). Plate XIII is extremely accurate and key diagrams have been added to facilitate its application. A table of wheel placements for maximum stress is also included. Plate XIII constitutes a complete and self-contained stress manual, and the writer recommends it as preferable to other tables or diagrams for obtaining stresses.

A graphic comparison of the approved M-loading with the Cooper loading and the composite loading is presented in Fig. 21. This diagram is similar to Fig. 6, which it supersedes. A comparison of Figs. 6 and 21 shows that the adopted modification of wheel spacing yields a desirable improvement of stress margin at short spans, at a slight sacrifice of scientific accuracy.

Fig. 22 presents a graphic comparison of the adopted M-loading with a modified loading formula which may be used as a "short-cut" for approximate results. This equation, for the M-10 loading, takes the simple form:

$$q = 700 + \frac{11\,000}{\sqrt{2\,l_1 + l_2}} \qquad (12)$$

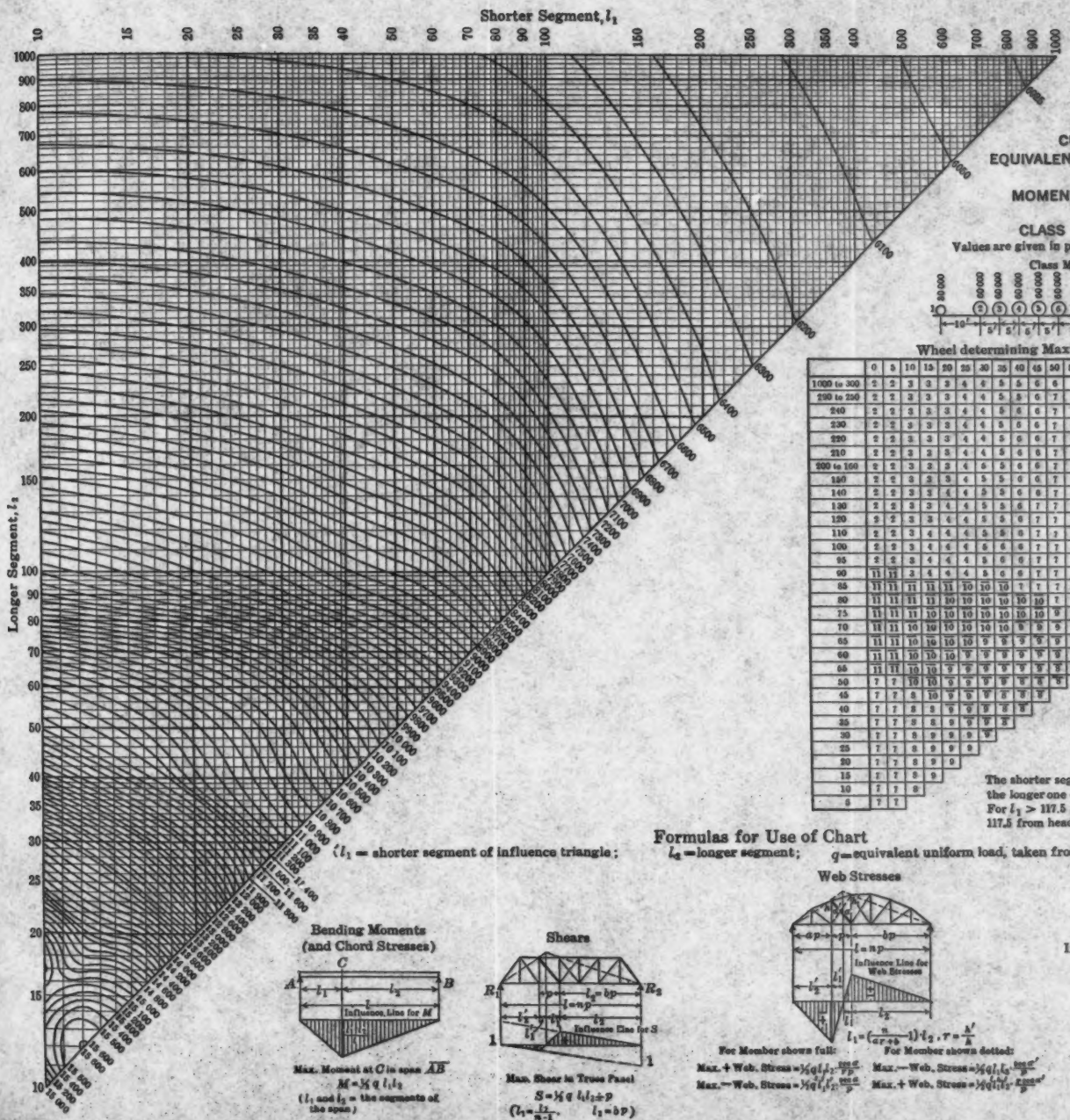
* See p. 517.

† See p. 500.

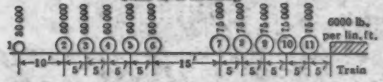
PLATE XIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXVI, No. 1520.
STEINMAN ON
LOCOMOTIVE LOADINGS
FOR RAILWAY BRIDGES.

CURVES OF EQUIVALENT UNIFORM LOADS FOR MOMENTS AND SHEARS FOR CLASS M-60 LOADING

Values are given in pounds per linear foot per track



Class M-60 Loading

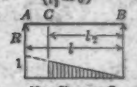


Wheel determining Maximum Stress

	0	5	10	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90	100	110
1000 to 300	2	2	3	3	3	4	4	5	5	6	6	7	7	8	8	9	9	9	10	10	11
290 to 250	2	2	3	3	3	4	4	5	5	6	7	7	7	8	8	9	9	9	10	10	11
240	2	2	3	3	3	4	4	5	5	6	7	7	7	8	8	9	9	9	10	10	11
230	2	2	3	3	3	4	4	5	5	6	7	7	7	8	8	9	9	9	10	10	11
220	2	2	3	3	3	4	4	5	5	6	7	7	7	8	8	9	9	9	10	10	11
210	2	2	3	3	3	4	4	5	5	6	6	7	7	8	8	9	9	9	10	10	11
200 to 190	2	2	3	3	3	4	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10
180	2	2	3	3	3	4	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10
170	2	2	3	3	3	4	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10
160	2	2	3	3	3	4	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10
150	2	2	3	3	3	4	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10
140	2	2	3	3	4	4	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10
130	2	2	3	3	4	4	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10
120	2	2	3	3	4	4	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10
110	2	2	3	4	4	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10	11
100	2	2	3	4	4	4	5	5	6	6	7	7	8	8	8	9	9	9	10	10	11
95	2	2	3	4	4	4	5	5	6	6	7	7	8	8	9	9	9	9	10	10	10
90	11	11	3	4	4	4	5	6	6	7	7	7	8	8	8	9	9	9	10	10	
85	11	11	11	11	11	10	10	10	7	7	7	8	8	9	9	9	9	9	10		
80	11	11	11	11	10	10	10	10	10	10	10	7	8	8	9	9	9	10			
75	11	11	11	10	10	10	10	10	10	10	9	8	8	9	9	9					
70	11	11	10	10	10	10	10	10	9	9	9	8	8	9	9						
65	11	11	10	10	10	10	9	9	9	9	8	8	9	9							
60	11	11	10	10	10	9	9	9	9	9	8	8	8								
55	11	11	10	10	9	9	9	9	9	8	8	8									
50	7	7	10	10	9	9	9	9	8	8											
45	7	7	8	10	9	9	9	8	8	8											
40	7	7	8	9	9	9	9	8	8												
35	7	7	8	8	9	9	9	8													
30	7	7	8	9	9	9	8														
25	7	7	8	9	9	9															
20	7	7	8	9	9																
15	7	7	9	9																	
10	7	7	8																		
6	7	7																			

The shorter segment is ahead followed by the longer one except where wheel is overlined
For $l_1 > 117.5$ place head of uniform load 117.5 from head of span.

Equivalent Uniform Loads for Shears and Reactions ($l_2 = 0$)



Max. Shear at C
$S = \frac{1}{2} q l_2 \frac{l_2}{l_1}$
Max. Reaction at A
($l_2 = l$)
$R = \frac{1}{2} q l$

l_1	q
1000	6612
975	6627
950	6643
925	6660
900	6678
875	6696
850	6715
825	6737
800	6759
775	6783
750	6808
725	6834
700	6861
675	6894
650	6926
625	6962
600	7000
575	7041
550	7085
525	7134
500	7187
475	7246
450	7310
425	7382
400	7462
375	7551
350	7652
325	7767
300	7898
275	8051
250	8230
225	8441
200	8696
175	9007
160	9220
150	9393
140	9574
130	9772
120	9996
110	10 227
100	10 485
95	10 620
90	10 926
85	11 273
80	11 625
75	11 973
70	12 306
65	12 604
60	12 917
55	13 149
50	13 500
45	14 149
40	14 906
35	15 796
30	16 833
25	18 000
20	19 750
15	20 000
10	22 500
5	30 000

Formulas for Use of Chart

(l_1 = shorter segment of influence triangle; l_2 = longer segment; q = equivalent uniform load, taken from chart for l_1 and l_2)

Bending Moments (and Chord Stresses)

Max. Moment at C in span $\overline{AB}$
$M = \frac{1}{2} q\, l_1 l_2$
(l_1 and l_2 = the segments of the span.)

Shears

Max. Shear in Truss Panel
$S = \frac{1}{2} q\, l_1 l_2 \div p$
($l_1 = \frac{l_2}{n-1}$, $l_2 = bp$)

Web Stresses

$l_1 = \left(\frac{n}{ar+b} - 1\right) l_2$, $r = \frac{h'}{h}$

For Member shown full:
Max. + Web. Stress $= \frac{1}{2} q\, l_1 l_2' \frac{\cos\alpha}{pb}$
Max. − Web. Stress $= \frac{1}{2} q\, l_1' l_2' \frac{\cos\alpha}{p}$

For Member shown dotted:
Max. − Web. Stress $= \frac{1}{2} q\, l_1 l_2' \frac{\cos\alpha'}{pb}$
Max. + Web. Stress $= \frac{1}{2} q\, l_1' l_2' \frac{\cos\alpha'}{p}$

Floorbeam Reactions Intermediate Pier Reactions



Max. Intermediate Reaction
$R = \frac{1}{2} q\, (l_1 + l_2)$
(l_1 and l_2 = the two adjoining spans.)

TABLE OF EQUIVALENT UNIFORM LOADS (PER TRACK) FOR DESIGNING RAILWAY BRIDGES

Based on Proposed Loading Formula M-60

$$q = 5000 + 60\,000\,\frac{Vl_2}{l_1 + l_2}$$

Shorter Segment l_1 — Longer Segment l_2

l_2 \ l_1	0	10	20	30	40	50	60	70	80	90	100	120	140	160	180	200	220	240	260	280	300	400	500	600	700	800	900	1000
1000	6897	6878	6860	6840	6823	6807	6789	6772	6757	6740	6724	6694	6664	6635	6608	6580	6555	6530	6506	6481	6459	6355	6264	6185	6117	6053	5998	5948
900	7000	6978	6956	6934	6915	6894	6875	6855	6836	6818	6800	6763	6730	6698	6666	6636	6608	6579	6551	6525	6500	6385	6285	6200	6125	6059	6000	
800	7120	7095	7070	7045	7020	6997	6973	6950	6928	6907	6885	6845	6806	6768	6731	6697	6663	6632	6600	6570	6543	6415	6306	6212	6132	6060		
700	7268	7238	7205	7175	7145	7115	7090	7060	7030	7010	6985	6935	6890	6845	6805	6765	6725	6690	6655	6620	6590	6440	6325	6220	6135			
600	7450	7410	7370	7330	7295	7260	7225	7190	7160	7130	7100	7040	6985	6930	6885	6830	6790	6750	6710	6670	6630	6470	6335	6225				
500	7680	7630	7580	7530	7480	7440	7395	7350	7310	7270	7235	7160	7095	7030	6970	6915	6860	6810	6765	6720	6675	6490	6340					
400	8000	7925	7860	7790	7725	7665	7610	7550	7500	7450	7400	7310	7220	7140	7070	7000	6935	6875	6820	6765	6715	6500						
300	8460	8350	8245	8150	8055	7970	7885	7810	7735	7660	7600	7475	7360	7260	7165	7080	7000	6925	6855	6790	6730							
280	8580	8460	8340	8235	8135	8040	7950	7865	7785	7710	7640	7510	7390	7280	7180	7090	7010	6930	6860	6790								
260	8720	8580	8455	8335	8225	8120	8025	7930	7845	7765	7690	7545	7420	7305	7200	7100	7015	6935	6860									
240	8875	8720	8575	8440	8320	8200	8100	8000	7905	7815	7735	7580	7445	7325	7210	7110	7020	6935										
220	9045	8870	8705	8560	8420	8295	8180	8070	7965	7870	7780	7620	7470	7340	7225	7120	7020											
200	9240	9040	8860	8690	8535	8395	8260	8140	8030	7925	7830	7650	7495	7355	7230	7120												
180	9475	9240	9030	8835	8660	8500	8355	8220	8100	7980	7875	7685	7520	7370	7240													
160	9740	9460	9215	8995	8795	8615	8450	8300	8160	8035	7920	7710	7530	7370														
140	10 070	9730	9440	9175	8940	8735	8550	8380	8225	8085	7955	7730	7535															
120	10 480	10 055	9695	9380	9110	8865	8650	8460	8285	8130	7990	7740																
100	11 000	10 450	10 000	9615	9290	9000	8750	8530	8335	8160	8000																	
90	11 325	10 695	10 175	9745	9380	9065	8795	8560	8350	8160																		
80	11 710	10 960	10 365	9875	9470	9130	8830	8575	8350																			
70	12 170	11 270	10 575	10 020	9560	9180	8860	8585																				
60	12 740	11 640	10 800	10 160	9650	9220	8870																					
50	13 480	12 070	11 050	10 300	9710	9240																						
40	14 490	12 590	11 325	10 420	9745																							
30	15 960	13 220	11 570	10 475																								
20	18 420	13 940	11 710																									
10	23 970	14 480																										



Any Triangular Influence Line,

l_1 = Shorter Segment of Influence Triangle.
l_2 = Longer Segment of Influence Triangle.
q = Equivalent Uniform Load taken from Chart for the values of l_1 and l_2 Values given in pounds per linear foot per track.

This table can be used for figuring the maximum stress in any member or section of any span. It gives stresses equal to (or slightly greater than) the maximum stresses producible by the heaviest existing locomotive loadings.

For Maximum Bending Moment at any section of any span, l_1 and l_2 = the two segments of the span.
$M = \frac{1}{2} \cdot q \cdot l_1 \cdot l_2$

For Maximum End Reaction or End Shear, $l_1 = 0$ and l_2 = the span.
$V = \frac{1}{2} \cdot q \cdot l_2$

For Maximum Shear at any section of a span, $l_1 = 0$. l_2 = the longer segment of the span and l = the span.
$V = \frac{1}{2} \cdot q \cdot l_2 \cdot \frac{l_2}{l}$

For Maximum Floorbeam Reaction or stress in Hanger, l_1 and l_2 = the adjoining panel lengths
$R = \frac{1}{2} \cdot q \cdot (l_1 + l_2)$

FIG. 20.

in which,

q = the equivalent uniform load, in pounds per foot.
l_1 = the shorter loaded segment, in feet.
l_2 = the longer loaded segment, in feet.

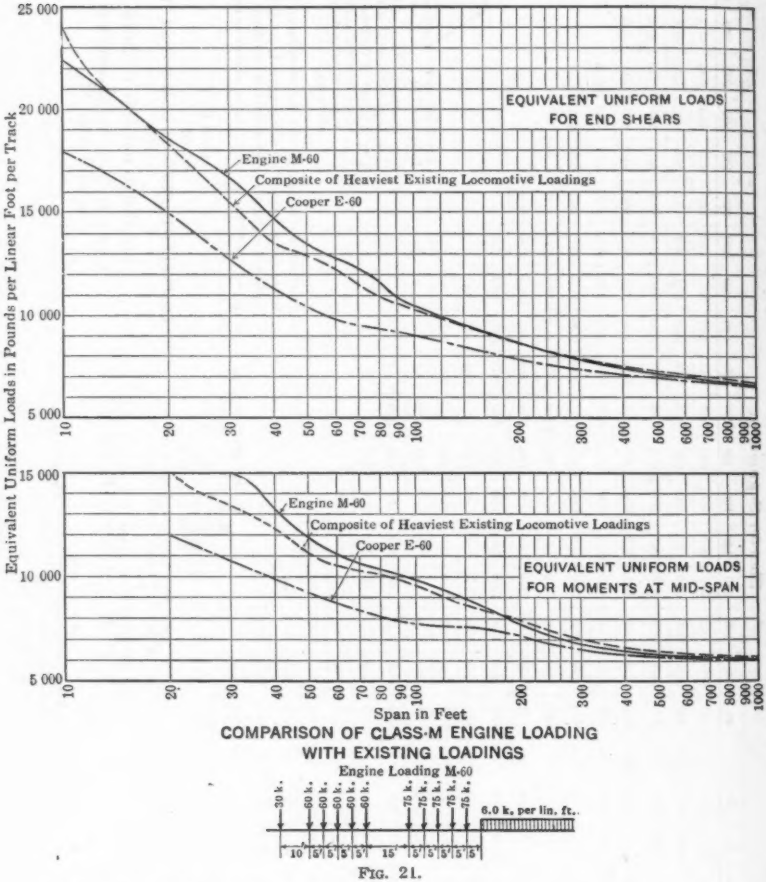


FIG. 21.

This loading formula reduces to:

$$q = 700 + \frac{11\,000}{\sqrt{\text{span}}}, \text{ for end shears} \quad (13)$$

$$q = 700 + \frac{11\,000}{\sqrt{l_2}}, \text{ for intermediate shears} \quad (14)$$

$$q = 700 + \frac{9\,000}{\sqrt{\text{span}}}, \text{ for mid-span moments} \quad (15)$$

The previous loading formula (Fig. 13) was developed independently of the typical engine diagram to approximate the composite loading; consequently, at some points, the respective variations of the two loadings (No. 1 and No. 3) from the composite loading were additive. In this discussion, the formula has been developed with the object of fitting the adopted M-engine diagram. Fig. 22 shows how closely this has been accomplished without sacrificing simplicity of coefficients. The coincidence could have been made closer by abandoning the use of round numbers in the formula. The introduction of the coefficient in the denominator permitted the independent adjustment of the moment and shear curves.

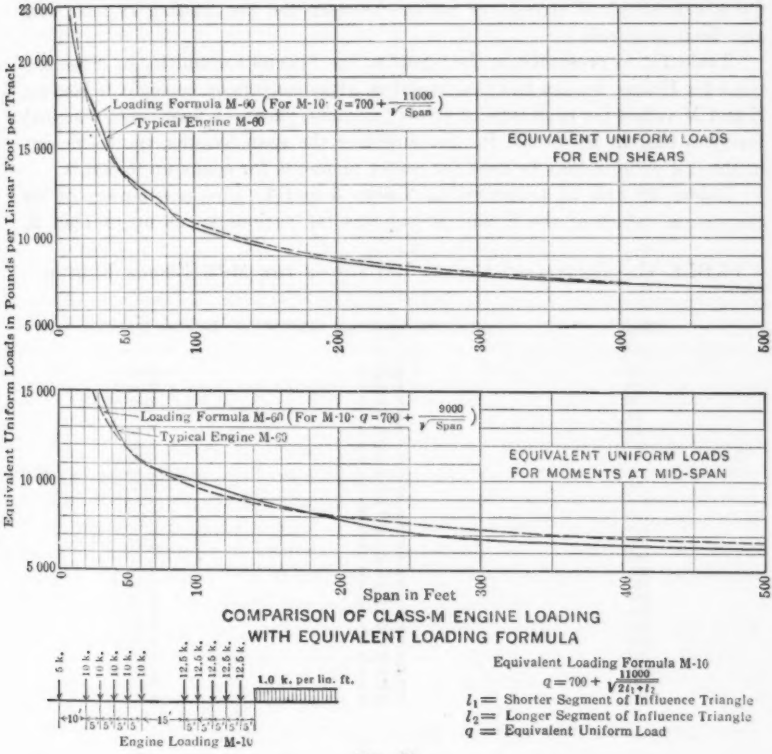


FIG. 22.

A table (Table 20*) of maximum moments, shears, and floor-beam reactions for the adopted M-loading has been presented by the Special Committee on Specifications for Bridge Design and Construction. Several of the discussors suggested the desirability of such a table to match the corresponding table which has been in use for the Cooper loading.

* See p. 519.

Other tables suggested in the discussions have been prepared for the adopted M-loading and also presented by the Committee. Table 21* gives the maximum bending moments at the various panel points of truss bridges for spans up to 450 ft., and Table 22† gives the shears in the various panels of truss bridges of spans up to 450 ft.

The argument that tables and diagrams are available for the application of the Cooper loading is nullified by the publication of the tables and diagrams for the M-loading just described. All tables and diagrams for the Cooper loading known to the writer are matched in form and scope by tables and diagrams for the M-loading. In addition, tables and diagrams which permit quick conversion of stresses or ratings between the two loading systems have also been prepared.

Table 17‡ as presented in the report of the Special Committee on Specifications for Bridge Design and Construction, gives conversion coefficients between E and M values for each type of stress (moment, shear, or floor-beam reaction) for span lengths up to 250 ft. By doubling the span lengths, the coefficients in the last column may be used for center moments for spans up to 500 ft.

Tables 11 and 12 (superseding Tables 3 and 4) give a summary of the conversion values of the E and M-loadings for various spans up to 1 000 ft.

TABLE 11.—EQUIVALENT COOPER RATINGS OF THE M-50 ENGINE LOADING.

Span.	E-rating for maximum moment.	E-rating for end shear.
10	E-62.5	E-62.5
20	E-62.5	E-62.5
30	E-68.5	E-66.8
40	E-67.8	E-65.9
50	E-65.4	E-64.5
60	E-64.2	E-66.1
70	E-63.6	E-65.0
80	E-64.4	E-62.4
90	E-64.6	E-59.7
100	E-64.4	E-58.3
120	E-61.5	E-57.5
140	E-59.2	E-56.9
160	E-57.3	E-56.4
180	E-56.1	E-56.0
200	E-55.2	E-55.4
250	E-53.3	E-54.7
300	E-52.5	E-54.2
400	E-51.4	E-53.3
500	E-51.0	E-52.7
600	E-50.6	E-52.4
700	E-50.5	E-52.0
800	E-50.4	E-51.8
900	E-50.3	E-51.6
1000	E-50.2	E-51.5

A conversion chart for changing E-ratings into equivalent M-ratings, for any point in any span up to 1 000 ft., is presented in Fig. 23. This diagram shows that E-60 is equivalent to M-45 at short spans and to M-60 at long spans.

The inverse chart for changing M-ratings into E-ratings is presented in Fig. 24. This diagram shows that M-60 is equivalent to E-81 at short spans, and to E-60 at long spans. For smaller indices, reduce the values in propor-

* See p. 520.
† See p. 526.
‡ See p. 516.

tion; thus, the equivalent of M-50 ranges from E-68 to E-50. With these tables and diagrams, the use of the M-loading is greatly facilitated.

Because of the great number of discussions received, it is practically impossible to answer them in detail, and it becomes necessary to confine this discussion to a review of the salient points that have been raised.

Although nearly all the discussions express a total or partial endorsement of the writer's conclusions, a number of dissenting views have been advanced.

TABLE 12.—EQUIVALENT M-RATINGS OF THE COOPER E-60 LOADING.

Span.	M-rating for maximum moment.	M-rating for end shear.
10	M-48.0	M-48.0
20	M-48.0	M-48.0
30	M-43.8	M-44.4
40	M-44.2	M-45.5
50	M-45.9	M-46.5
60	M-46.7	M-45.4
70	M-47.1	M-46.2
80	M-46.6	M-48.1
90	M-46.4	M-50.2
100	M-46.6	M-51.5
120	M-48.8	M-52.2
140	M-50.6	M-52.7
160	M-51.5	M-53.2
180	M-53.5	M-53.6
200	M-54.3	M-54.1
250	M-56.3	M-54.8
300	M-57.1	M-55.3
400	M-58.4	M-56.3
500	M-58.8	M-56.9
600	M-59.4	M-57.2
700	M-59.5	M-57.7
800	M-59.6	M-57.9
900	M-59.7	M-58.1
1 000	M-59.8	M-58.3

The discussions of this group make frequent reference, as authority for their position, to the decision of the American Railway Engineering Association in 1920 to retain the Cooper loading, based on studies presented in a Sub-Committee report by Messrs. Welty and Bouton.* Those studies were based on comparisons of modern locomotives and train loads with the Cooper standard, from which the authors arrived at the conclusion that the Cooper loading adequately covered the locomotives assumed.

The writer, assuming substantially the same locomotives and train loads, and following the reasoning outlined in his paper, arrives at the conclusion that the Cooper loading is unsatisfactory as a present-day specification and that a better loading can be devised.

In attacking this conclusion, the principal argument of the dissenting group is that the writer's study is based on extra heavy locomotives, not in common use. In reply, it may be pointed out that, with one exception, the locomotives considered by the writer are identical with the heavy locomotives considered in the report of the Sub-Committee of the American Railway Engineering Association, referred to previously. The assumptions are the same, but the conclusions are different. Those who endorse the report mentioned cannot consistently attack the writer's loading assumptions.

* *Proceedings*, Am. Ry. Eng. Assoc., January, 1920, pp. 511–520.

The writer would defend his loading assumptions, even if there had been no precedent to guide or justify him. He hoped he had made the logic of his argument clear. He based his studies on the heaviest existing locomotives, because they are the best index available of probable future development. History, in this respect, has repeated itself so often that it seems a lack of foresight to regard the present locomotive development as the limit of growth. Mr. Loweth, in his discussion, recalls the frequent arguments in the past that the limit of locomotive development had been reached, and writes: "Each

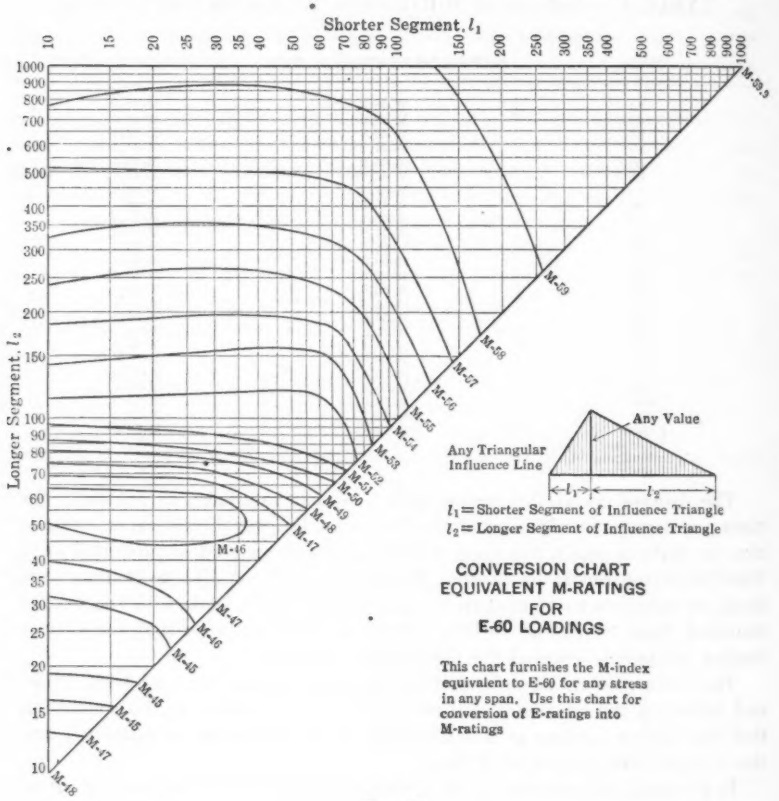


FIG. 23.

new locomotive seemed to be about the maximum that could be designed and would be practicable; yet, as the years passed, there was a continuing succession of heavier locomotives, and the end is not yet in sight." The extra heavy locomotives of yesterday have become the average locomotives of to-day; and the extra heavy locomotives of to-day will be the average locomotives of to-morrow. In the past, the life of bridges has been curtailed by loading augmentation; engineers should not be guilty of repeating that mistake. Bridges should

be proportioned to the loads of 25 or 30 years hence, not to the loads prevailing to-day. Loading diagrams should conform, as far as possible, to the expected future loads. Specifications should be progressive, not short-sighted. Good engineering in this field demands vision, based on recognition of the lessons of the past.

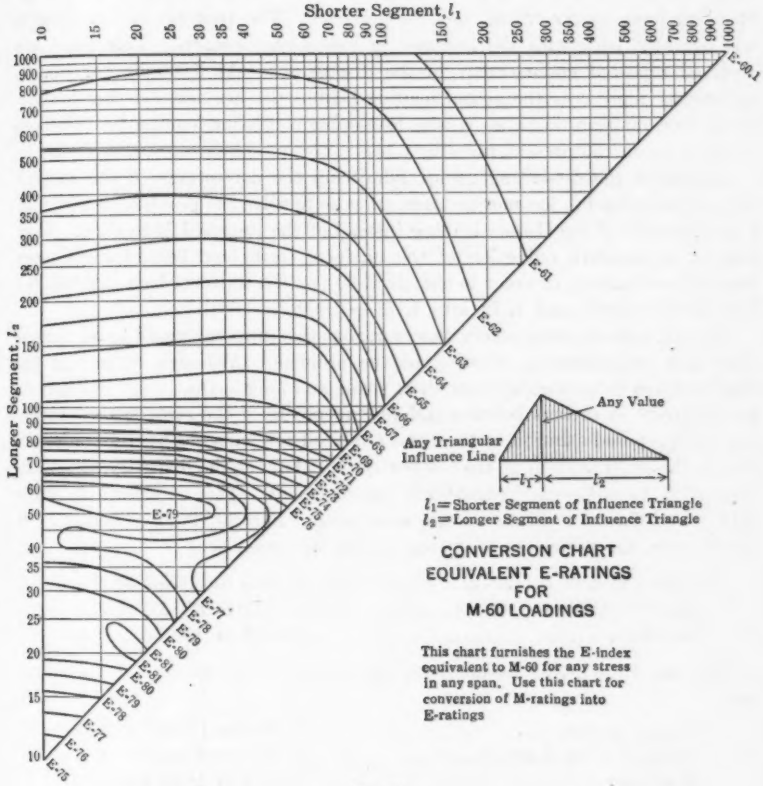


FIG. 24.

The next criticism by the dissenting group is on the assumption of train loading behind the locomotives entering into the composite loading. Influenced by their use of the Cooper loading, they contend that 6 000 lb. per lin. ft. is too light a uniform load to follow these locomotives. The answer is simple: The actual train loads following these locomotives scarcely equal and certainly do not exceed 6 000 lb. per lin. ft. In the report of the Sub-Committee of the American Railway Engineering Association, previously referred to, the train load assumed behind the same locomotives consisted of 210 000-lb. cars, amounting to only 4 770 lb. per lin. ft. The writer has been more generous in this regard than the authors of the report which is being cited as an

authority. On consulting the railroads in question, it will be found that the heaviest locomotives are used for hauling trains of cars weighing less than 6 000 lb. per lin. ft. Trains weighing 4 770 lb. per lin. ft. are assumed by the Pennsylvania Railroad Company behind its heaviest locomotives. The trains behind the heaviest locomotives of the Michigan Central Railroad weigh only 4 000 lb. per lin. ft. The locomotives are made heavy in order to haul longer trains, not heavier cars. The increase in car weights has not kept pace with the increase in locomotive weights. This point has received repeated endorsement in the discussions. As 6 000 lb. per lin. ft. generously represents the actual loading behind the heaviest existing locomotives, there appears to be no reason to assume a heavier loading behind those locomotives in the present investigation.

Others of the dissenting group, admitting the correctness of the writer's ratio of train load to locomotive loads, propose to solve the problem by adopting a modification of the Cooper loading instead of the proposed M-loading. They suggest a one-sixth reduction of the uniform train load behind the Cooper locomotive diagram, in order to change E-60 (with a uniform load of 6 000 lb.) into E-60 + 5 000, and E-72 into E-72 + 6 000.

Careful consideration shows, however, that this proposal would be an incomplete and unsatisfactory solution of the problem. Although correcting the disproportion in ratings between short spans and long spans, it will not correct the disparity in ratings between different members of the same span. Applying this proposed solution to the truss analyzed for purposes of illustration in Fig. 1, this modification of the Cooper system changes the disparity of ratings from 37% to 36%—an insignificant improvement. This is shown on Plate XIV for different loadings on the same span. For the Pennsylvania N-1-S locomotives, the variations in rating within this truss are:

Cooper system ..... E -54.7 to E -75.0 = 37%
Modified Cooper system..... E′-55.0 to E′-75.0 = 36%
M-system ..... M-46.2 to M-55.6 = 20%

For the Erie 2-10-2 locomotives, the variations in rating for this truss are:

Cooper system ..... E -53.9 to E -68.4 = 27%
Modified Cooper system..... E′-54.2 to E′-68.4 = 26%
M-system ..... M-45.6 to M-50.6 = 11%

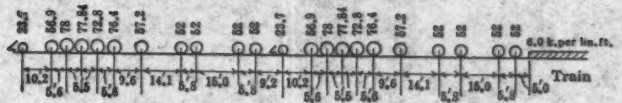
For the Pennsylvania I-1-S locomotives, the variations in rating within the 140-ft. span are:

Cooper system ..... E -53.5 to E -67.6 = 26%
Modified Cooper system..... E′-53.9 to E′-67.6 = 25%
M-system ..... M-45.4 to M-50.4 = 11%

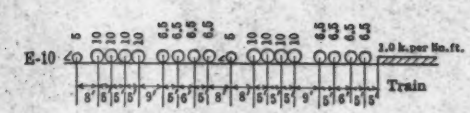
Averaging the results for the three locomotives, the following values are obtained:

Cooper system ..... Average discrepancy = 30%
Modified Cooper system..... Average discrepancy = 29%
M-system ..... Average discrepancy = 14%

RATINGS FOR TRUSS DESIGNED FOR

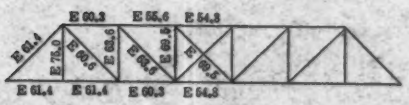


2 PENNSYLVANIA N1S ENGINES
(SANTA FÉ)



COOPER LOADING

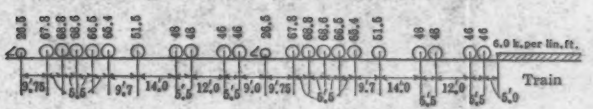
E-RATING



Max. Rating = E 75.0 Min. Rating = E 54.7

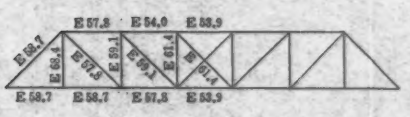
Variation = 37%

RATINGS FOR TRUSS DESIGNED FOR



2 ERIE ENGINES 2-10-2
(SANTA FÉ)

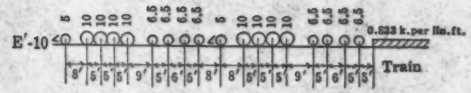
E-RATING



Max. Rating = E 68.4 Min. Rating = E 53.9

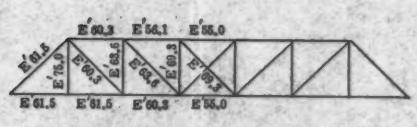
Variation = 27%

E-I



MODIFIED COOPER LOADING

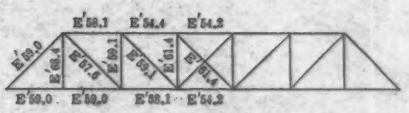
E'-RATING



Max. Rating = E'75.0 Min. Rating = E'55.0

Variation = 36%

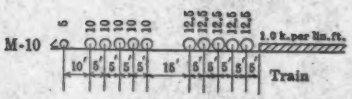
E'-RATING



Max. Rating = E'68.4 Min. Rating = E'54.2

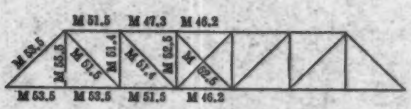
Variation = 26%

E'-I



CLASS M LOADING

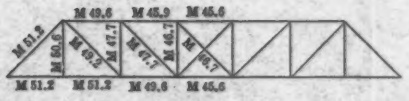
M-RATING



Max. Rating = M 55.6 Min. Rating = M 46.2

Variation = 20%

M-RATING



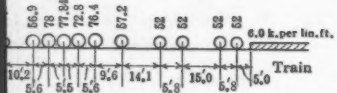
Max. Rating = M 50.6 Min. Rating = M 45.6

Variation = 11%

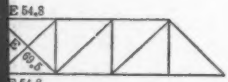
M-I

COMPARISON OF VARIATIONS OF RATING IN A 140-FT. SPAN FOR DIFFERENT LOADING SYSTEMS

RUSS DESIGNED FOR

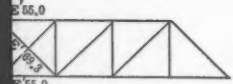


NIA N1S ENGINES
NTA FÉ)



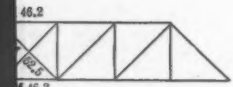
Min. Rating=E 54.7

tion=37%



Min. Rating=E'55.0

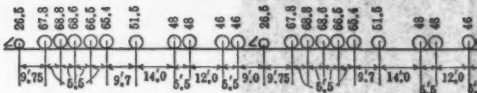
tion=36%



M¹n. Rating=M 46.2

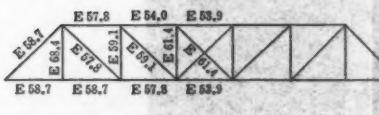
ion=20%

RATINGS FOR TRUSS DESIGNED FO



2 ERIE ENGINES 2-10-2
(SANTA FÉ)

E-RATING



Max. Rating=E 68.4 Min. Rating=E 5

Variation=27%

E'-RATING



Max. Rating=E'68.4 Min. Rating=E'5

Variation=26%

M-RATING



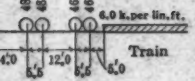
Max. Rating=M 50.6 Min. Rating=M

Variation=11%

COMPARISON OF VARIATIONS OF RATING IN A 140-FT. SPAN FOR DIFFERENT LOADING SYSTEMS

PLATE XIV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LXXXVI, No. 1520.
STEINMAN ON
LOCOMOTIVE LOADINGS
FOR RAILWAY BRIDGES.

GNED FOR



0-2



ating—E 53.9



ating—E'54.2



Rating—M 45.6

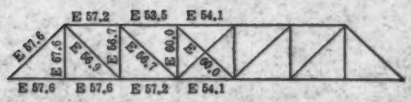
AN

RATINGS FOR TRUSS DESIGNED FOR



2 PENNSYLVANIA 11S ENGINES
(DECAPOD)

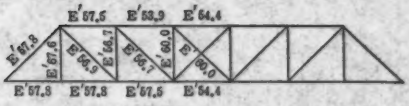
E-RATING



Max. Rating—E 67.6 Min. Rating—E 53.5

Variation—26%

E'-RATING



Max. Rating—E'67.6 Min. Rating—E'53.9

Variation—25%

M-RATING



Max. Rating—M 50.4 Min. Rating—M 45.4

Variation—11%



In other words, the proposed modification of the Cooper loading reduces the disparity of ratings within a given span by only 1%, whereas the adoption of the M-loading would reduce the error to less than one-half. These comparisons (Plate XIV) show that the suggested modification of the E-system does not solve the problem of disparity of ratings within a span. Neither will it correct the disparity of rating between two short spans, for example, between 40 ft. and 140 ft. In addition to this defect, the proposed modification of the Cooper loading would not avoid a revision of existing stress tables and ratings; and this revision would be more likely to cause error and confusion than a clean-cut change to a new system.

Once the disproportion of the standard Cooper system is recognized, the remedy is not to "patch up" that system with inadequate modifications, but to make a fresh start with a simpler one founded on modern heavy loadings and, therefore, better designed to cover future developments. If the M-system is admitted to be simpler and better than the E-system, engineers should not stop half way in correcting their standards, but should make a clean-cut change and thereby secure all the advantages of the new standard.

Another argument advanced against the proposed loading is that the majority of existing bridges are of spans of less than 100 or 120 ft., and that for these short spans the Cooper system yields a sufficiently close approximation to the stress ratios of modern loadings. This argument might have weight if the specifications were limited to spans of this length. It is true that if all spans were less than 80 or 100 ft., it would make comparatively little difference what system of loading specifications was used; but specifications are not limited to plate-girder spans. The A. R. E. A. Specifications include 300-ft. spans; the new specifications of the Special Committee on Specifications for Bridge Design and Construction, of the Society, are intended to apply to bridges of any span. If the Cooper system is satisfactory for plate-girder spans, but gives rise to serious discrepancies in longer spans, it should certainly be superseded by a loading system that gives consistent and harmonious results at all spans.

With the exception of the dissenting group, the arguments of which have just been reviewed, the general reaction to the writer's investigation and conclusions has been that of confirmation and endorsement. It is well expressed by the following quotation from the discussion by Mr. Strachan:

"* * * the author's determination of M-60 after careful study of the stress-producing effects of the heaviest existing locomotives, and his subsequent building up of a series of M-diagrams bearing simple ratios to M-60, in the manner of the Cooper loading, is an important step in the direction of introducing a scientific and modern standard load system for the economical design of railroad bridges and for the rating or comparison of their carrying capacities; a system of which the need is admittedly great in consequence of the changes in locomotive construction which have taken place since the Cooper system of loading was devised."

The individual discussions will now be reviewed in the order of their publication.

The criticism of Mr. Purdon, that the engines considered in the investigation are "enormous" or "exceptional" has already been answered. Even if all existing locomotives had been included in the analysis, the results would have been the same, as the object of the analysis was to determine the enveloping or maximum stress effects. No specification can be regarded as scientific or adequate unless it takes into account maximum stress-producing effects. Mr. Purdon's suggestion that bridges should provide for possible small engines "as a minimum" does not appear to be logical.

Mr. Purdon makes the point that the proposed typical engine (M-Loading No. 1) "does not resemble an engine". That, of course, is a matter of personal opinion. If the M-diagram does not resemble any particular engine, that is because it is a composite of different types, conventionalized and simplified. It combines the governing features of the Santa Fé or Decapod with those of the Mallet locomotive. As stated by Mr. Cochrane:

"Modern locomotives differ so greatly in the arrangement and spacing of axle loads that no single type can be found to represent all of them, unless it is a more or less conventional system, such as the author's first alternative loading. A composite loading derived from a number of different types of heavy locomotives will undoubtedly be more nearly representative of future loadings than any single loading series, and, consequently, will make more adequate provisions for future developments in locomotive design."

The point is also clearly covered by Professor Bird.

The proposed M-loading diagram may be regarded as an outline of a Mallet engine with different weights on the two sets of axles (for which there are precedents); or, it may be regarded as a Santa Fé or Decapod locomotive with the engine placed next to the train (for maximum stress). Resemblance to any particular type could easily be secured by minor modifications; but it would be at the expense of simplicity and symmetry.

Mr. Purdon attempts to demonstrate the adequacy of the E-loading by showing that a certain span designed for the Cooper loading gives satisfactory uniformity of stress under an E-42 double-headed Consolidation engine. This was naturally to be expected, as the Cooper loading diagram is a double-headed Consolidation engine. It is for types other than Consolidation engines, and for loads exceeding E-50, that the Cooper loading reveals its inadequacy.

Mr. Purdon gives figures showing the rapidly growing use of the Santa Fé type of locomotive, although its efficiency only slightly exceeds that of the Consolidation engine. He also shows the much greater efficiency of the Decapod or "ten-wheeler". In these conclusions, he is confirmed by Mr. Bland; but they serve only to strengthen the writer's position. Whether the Santa Fé or the Decapod locomotive represents the governing type in the future is immaterial in this discussion, as their stress-producing effects are practically identical; the proposed M-loading will provide for them, whereas the E-loading does not fit either. Plate XIV shows how the M-loading gives much better results for Santa Fé and Decapod locomotives than the Cooper loading. For the Decapod locomotive, the E-rating variation in a given span is 26%, whereas the M-rating variation is only 11 per cent.

Mr. Strachan points out the inconsistent accuracy of extreme precision in bridge designing when the fundamental loading assumptions are wrong. He characterizes the Cooper system as "an ancient generalization, which, although excellent in its day, has outlived its usefulness." He endorses the proposed M-loading, and states that "its adoption would tend toward augmented economy and the elimination of many unprofitable complications inseparable from the use of an obsolete load specification." Although preferring the typical engine diagram (M-Loading No. 1) as a load specification, Mr. Strachan suggests the use of the alternative loadings as "short-cuts" under suitable circumstances. This idea has been applied in Fig. 22 in which is presented a simple loading formula that may be used as an approximate substitute for the M-engine loading specification.

Mr. Cochrane supports the writer in his claim that the Cooper loading suffers from lack of balance between the locomotive axle concentrations and the uniform train load. He points out that, in the Cooper series, the maximum axle load is ten times as great as the train load per linear foot, whereas, in the case of the heaviest loadings now in use, this ratio is about 13. (This ratio for the Pennsylvania Railroad H-C-1-S locomotive, attains 14 or 15). Mr. Cochrane, therefore, agrees with the writer in the desirability of making the axle loads heavier than those of the Cooper series in proportion to the uniform train load.

For specification purposes, Mr. Cochrane expresses a preference for the loading formula (M-Loading No. 3) as being the simplest and most direct. He suggests, however, the use of an engine diagram (M-Loading No. 1) in conjunction with the formula as an approximate depicture of the loading. To reduce the discrepancy between the two alternative loadings, he proposes some modifications of the formula; but the writer submits Fig. 22 as a simpler solution of the problem.

Mr. Lucius predicts that the tendency in future bridge loadings will not be toward increased driver axle loads, but rather toward an increase in the number of drivers. This is an argument for the adoption of the M-system in preference to the E-system, as the increase in the number of drivers means the abandonment of the Consolidation type of locomotive for the more modern types on which the M-loading is based.

Mr. Lucius also raises the question of electrification and the new motor diagrams which it may develop. This is a question in regard to which it is difficult to come to any conclusion or to make any predictions at this time, as the standardization and development of electric motor diagrams is still in its infancy. Nevertheless, a study of the tendency in recent heavy electric locomotive diagrams indicates that the M-system will be better fitted than the E-system to the electric locomotives of the future.

Mr. Worthington emphasizes the fact that the present locomotive types have outgrown the Cooper loading diagram, and endorses the writer's conclusion that a material revision of this loading diagram is necessary in order to secure uniformity of stress and economy of metal. After a care-

ful study in conjunction with H. F. Passel, M. Am. Soc. C. E., he expresses a preference for the typical engine diagram (M-Loading No. 1).

Mr. Gilbert also confirms the writer in his claim that the Cooper loading does not suitably provide for modern types of engines. From his own investigations, Mr. Gilbert believes that a diagram of the Santa Fé type would properly represent modern locomotive loadings. He points out the increasing use of the Santa Fé type, and states, as his conclusion, that the Santa Fé is a permanent type which should be given full consideration in selecting a proper bridge loading. This conclusion appears to be incontrovertible. The writer would point out that the M-loading diagram contains the essential elements of the Santa Fé axle diagram, conventionalized and simplified; and he would call attention to the comparison previously mentioned, which shows how the M-loading is superior to the Cooper loading in representing the stress-producing effects of Santa Fé locomotives.

Mr. Gilbert characterizes the proposed M-60 loading as "equal to or greater than that of any locomotive now in service", and questions whether railroad executives will approve of such a heavy loading. The writer is convinced that the adoption of the present maximum (M-60) for the design of new bridges on main lines will be amply justified by the inevitable increase in loading during the life of those structures. However, if any railroad officials regard this loading as excessive, there is nothing to prevent them from using M-50 or M-55 instead; or, preferably, the unit stresses may be augmented so as to give the same weight of structure with M-60 loading.

In the proposed M-60 loading, each of the five heavy axle loads are 75 000 lb. Mr. Gilbert shows by tabulated data that average axle loads exceeding 60 000 lb. are common, and that axle loads of 78 000 lb. are in use. To this may be added the Pennsylvania H-C-1-S engine, which has axle loads of 86 400 lb. Mr. Gilbert admits that the tendency of driver axle loads is to increase (on account of the increasing demand for tractive power). In view of these facts, a loading specification (M-60) with 75 000 lb. on the driver axles, does not seem to be excessive if bridges are to be designed with any regard to future requirements.

Mr. Gilbert shows by diagrams (Fig. 15) the inadequacy of E-60 loading to provide for actual engines; he finds the resulting over-stress produced by Santa Fé locomotives in the shorter spans to be approximately 25 per cent. This would correspond to E-75 which is equivalent, at these spans, to M-60. Accordingly, Mr. Gilbert's diagrams demonstrate that M-60 is not too high a loading to protect new designs against present-day locomotives.

Mr. Gilbert emphasizes the importance of a loading specification which will give ample section to short spans, so as to provide the needed protection against corrosion to floor systems, hangers, etc. This requirement is fulfilled by the M-system as it corrects a relative deficiency of 25% in the Cooper loading for short spans.

Mr. Waddell generously endorses the writer's thesis. He discounts the argument for retaining the Cooper loading as a rating standard, declaring that the M-loading system provides just as convenient a rating standard.

Mr. Waddell expresses a preference for M-Loading No. 3 (the loading formula) as the most scientific and convenient; and he endorses the suggestion made by Mr. Cochrane that M-Loading No. 1 (the typical engine diagram) be used in conjunction with the formula as an approximate representation.

Mr. Lindenthal properly characterizes the subject as one in which meticulous accuracy is out of place. He expresses his preference for M-Loading No. 1 (the typical engine diagram), as it would represent the smallest departure from present methods, merely substituting a new and simpler wheel-load diagram for the Cooper diagram, and as it is sufficiently inclusive of the maximum effects of loads from any of the seven locomotive-wheel diagrams illustrated in the paper.

Mr. Lindenthal points out that the floor construction, consisting of short spans, suffers more than the trusses from an increase of locomotive weights. That is one of the writer's reasons for advocating a loading specification which will be relatively more generous to short spans than the Cooper specification. Mr. Lindenthal mentions the greater allowance for impact in floor members, but he confuses the issue by suggesting that this impact allowance covers to a certain extent unforeseen increases in wheel loads. The writer is of the opinion that the impact provision for short spans in present specifications is inadequate and cannot be considered as including a margin for load increase.

Mr. Lindenthal raises the question of relative impact (due to difference in speed) of freight and passenger locomotives. This point must await further experiment and, when determined, should be covered in the specification for impact. The writer does not believe, however, that light passenger locomotives at full speed will produce greater total stresses than the heavy axle loads at freight-train speed; in addition, the possibility of the heavy locomotives running over a bridge at full speed when relieved of train load, must be considered as well as the possibility of passenger trains being limited in speed in going over the structure. The subject of impact provision is a large problem in itself, and should not be permitted to obscure the issue in the present investigation. Moreover, even if Mr. Lindenthal is correct in his contention that passenger locomotives should govern in design, it does not affect the choice between the M-loading and the Cooper loading, as the trains behind passenger locomotives are much lighter than those behind freight locomotives, and the ratio of train weight to locomotive weight in passenger engines conforms to the M-loading and not to the E-loading. For example, the K-4-S passenger locomotive of the Pennsylvania Lines, with its following train load, is found to be closely and consistently represented by M-49.

Professor Caughey endorses the writer's thesis that the Cooper loading should be replaced by a loading more in agreement with the weights and axle groupings of modern railway rolling stock. He records his preference for M-Loading No. 1 (the typical engine diagram) and gives some cogent reasons for this choice.

Professor Caughey raises the question of train loads in exceptional cases being heavier than the engine, when light locomotives are used with short

trains of heavy cars. Any such exceptional case should receive special study in adopting a design load; if a suitable index is used, either the M-loading or the E-loading will give safe stresses for this case.

The writer is grateful to Mr. Ralston for his expressive review of the questions under consideration, and for his encouraging words of appreciation. Mr. Ralston endorses the writer's conclusions as to the erratics of the Cooper loading system, and advocates the adoption of the M-system as consistent and logical. He gives a striking review of the history of locomotive development, and records his conviction that the M-loading system will remain applicable for a long time to come.

Mr. Bland, in his discussion previously mentioned, contributes an interesting account of the introduction of the E-loading system. He recalls that the late Theodore Cooper, M. Am. Soc. C. E., gave the E-25 loading as a minimum, and the E-40 as a maximum, which latter he regarded as the heaviest loading possible. Among the disadvantages of the Cooper loading, Mr. Bland cites the incorrect axle-load ratios and distances, the fixing of a ratio between train load and axle load which was not applicable to loads above E-40 or E-50, and the inapplicability of the E-standard to engines other than those of the Consolidation type.

From considerations of locomotive development, Mr. Bland concludes that the E-loading is not applicable beyond Cooper's upper limit of E-40, and that, for many years, the Cooper loading has ceased to be a proper standard.

After condemning the inconsistencies resulting from the use of the Cooper loading (such as the necessity for different E-ratings for different parts of the same bridge), and recording his hope that the M-loading in one of its three forms will soon be adopted by the Profession, Mr. Bland expresses his own preference for M-Loading No. 3 (the loading formula) for bridge design. The writer is in sympathy with this preference for the loading formula, but regrets that conditions do not appear ripe for this innovation in loading specifications. If M-Loading No. 1 (the typical engine diagram) is adopted as a general specification, the individual designer may use a loading formula (either that of Fig. 13 or Fig. 22) as a sufficiently close equivalent.

Mr. Bland endorses the writer's underlying principle of a composite loading, and recommends that each railroad "consider the group of its engines for which it wishes to provide, form a 'composite' of such engines, and then choose the M-loading (taking into account future development) that best fits its case, and use such loading in its specifications." According to his correspondence with the writer, Mr. Bland has done this for the engines of the Pennsylvania System, and has established a series of M-loadings which give consistent protection to the various types of locomotives in use.

Based on considerations of comparative efficiency, Mr. Bland makes a prognosis of future engine development. He shows that the development of the Mikado type (2-8-2) is a step in the wrong direction, but that the Decapod type (2-10-0) offers the greatest advantages. The I-1-S engine (Decapod) yields 69% more tractive power than the H-10-S (Consolidation), with an increase of only 45% in the weight of the engine. Accordingly, he concludes that the Decapod type is more likely to be increased in the future than other

types. As the stress effects of the Decapod and Sante Fé types are practically the same, either may be considered as a governing type to which a correct loading specification should conform in order to provide for the most probable future development.

Mr. Johnson admits the defects of the Cooper loading as applied to roads using the heavy locomotives of the present day. He points out, however, the great mileage of roads on which lighter locomotives prevail, and he considers the Cooper loading satisfactory for such lines. In reply, the writer would state that the M-loading (with the index reduced from M-60) would be just as satisfactory as the E-loading for these lines, and a loading standard (like the M-series) which can be used on all lines, both heavy and light, is obviously preferable to one which is limited in application to second-class roads.

Furthermore, Mr. Johnson admits that Santa Fé locomotives are a governing type on these lighter roads, and the writer has previously shown that the M-loading gives more consistent protection than the E-loading to engines of the Santa Fé type. It may also be noted that the secondary lines referred to generally have steeper grades and sharper curves, requiring heavier locomotives in proportion to the weight of the train; and the M-system, with its correction of ratio of engine weight to train weight, will fit this loading better than the E-system.

Mr. Johnson admits that the proposed M-loading is in the right direction, but thinks its application to the lighter lines of the country should be deferred. As opposing this view may be urged the desirability of a uniform standard for all lines. If the advantages of the M-system for the main lines are admitted, considerations of uniformity would argue for the adoption of that loading system for all lines. M-50 or M-60 can be used on the main lines and M-40 or M-50 on the lighter lines, depending on the governing types and weights of locomotives.

Mr. Rogers admits that the M-60 loading covers all stresses in all spans from the five heaviest present-day types of engines. He adds that the proposed conventional loading would conform admirably to the requirements if all these five types of locomotives were in use on any one railroad; otherwise, he argues, there will be a waste of material put into bridge structures to suit the requirements of engines of foreign roads. In this argument, Mr. Rogers misses the underlying principle of the writer's paper, namely, that a composite loading is the most scientific solution of the problem of devising a single standard to fit all engine types with a view to future development. Such comparisons as those illustrated previously show that the M-loading (based on the composite) yields sufficiently close agreement with individual engines entering into the composite; certainly a much closer agreement than that provided by the Cooper loading. Inasmuch as disproportion of stress means waste of material, the comparisons given demonstrate that designs made with the M-loading will involve less waste than if the E-loading is used.

Mr. Rogers also raises the point that lighter engines are not reduced models of the heavy locomotives which entered into the composite. The fact is,

however, that although the M-loading may not be a correct pictorial representation of these lighter locomotives, it will be found to yield correctly proportioned designs. Mr. Bland, who has made special studies along this line, has found the M-loadings to give uniform and consistent results for the lighter engines on the Pennsylvania Lines; for instance, he found that the Mikado and Consolidation locomotives are correctly and consistently represented by M-50 as a design load. (According to Mr. Bland's graphs and conclusions, the L-1-S Mikado is economically protected by M-50; the K-4-S passenger locomotive by M-49; the H-10-S Consolidation by M-47; the H-8a-S by M-45; and the H-6b-S by M-42.)

Mr. Rogers adds the argument that if the locomotives on any line reasonably conform to the Cooper loading, the use of the M-loading would involve a waste of metal at short spans. The answer to this is the fact that the disproportion between long and short spans arises mainly from the incorrect ratio of train load to locomotive loads in the Cooper system. A change in type of locomotive does not alter this condition. With proper reduction of the index, the M-loading will give consistent results for the conditions assumed by Mr. Rogers. However, if there is any excess of metal in short spans resulting from the use of the M-loading, that extra metal is in a good place. A loading that adds extra metal to floor systems and short spans is unquestionably preferable to one which skimps the short spans and wastes metal on the long spans.

Mr. Rogers recommends that instead of adopting the M-loading with varying index, each railroad should devise its own special loading for design. The writer submits that this would result in confusion of ratings and comparisons, and would be far from a solution of the problem.

Mr. Stone recommends the new loading system as a step in the direction of filling a recognized need. He expresses a preference for M-loading No. 1 (the typical engine diagram), because, with suitable tables, it would be as simple to apply as any other method. He suggests the appointment of a committee to consider the question of endorsing the proposed M-loading. This has already been done by the Special Committee on Specifications for Bridge Design and Construction.*

Professor Bird, in his discussion previously mentioned, endorses the writer's conclusions and reports that he has calculated a number of designs using the M-loading, and has been pleased to note how well the values fitted the maximum moments and shears producible by actual locomotives.

He refers to the charts of equivalent uniform loads devised by the writer (Plates IV, VI, XI, and XIII) and suggests that it would be well to explain and illustrate the use of these charts with typical problems. Instead of taking space in this discussion, the writer would refer the readers to the explanation and illustration of these charts published elsewhere.†

Professor Bird also suggests that tables of moments and shears for the M-loading would be a helpful addition to the paper. Such tables were also

* See pp. 472, 477.

† *Engineering News*, April 22, 1915; "American Civil Engineer's Handbook," 1920, pp. 849, 853, 854, 884, 904, 906 and 908; and *Lefax*, September, 1920, No. 10–348.

prepared by the writer at the request of the Special Committee on Specifications for Bridge Design and Construction and are incorporated in the report of that Committee.*

Professor Bird endorses the idea suggested by Mr. Seaman, to include both the E-loading and the M-loading in the specifications, in order to make the change optional during the transition period.

Professor Fuller, in his discussion, endorses the writer's conclusions. "The Cooper loading", he states, "no longer represents even to an approximate degree the modern locomotives and trains."

Professor Fuller declares his preference for M-Loading No. 1 (the typical engine diagram), and raises the interesting question of the flexibility of the loading formula (M-Loading No. 3), suggesting that re-adjustment may be found necessary after the M-loading has been in use half a century or more. In reply, the writer would point out that the loading formula (Equation (4)) contains two numerical coefficients; by modifying one or both of these coefficients, the curves of equivalent uniform loads in Fig. 13 may be raised or changed in slope to conform to any requirements. In the modified form of the loading formula (Equation (12)), there are three numerical coefficients, giving still greater flexibility. By modifying the coefficient of l_1 in the denominator, the curve for center moments may be adjusted independently of the curve for shears. Fig. 22 is a graphic demonstration of the flexibility of this loading formula, as curves have been obtained to coincide so closely with the arbitrary curves of a typical engine diagram. The coincidence could have been made still closer by deviating from the round numbers chosen for the three coefficients in the formula.

Professor Bishop emphasizes the conclusion that the Cooper loading has outlived its usefulness and should be replaced. After calling attention to the disproportion and lack of simplicity of the Cooper loading, Professor Bishop expresses a fundamental principle when he states: "Any new loading should give approximately the same results as the actual loads which are likely to be used in the near or distant future." In this connection, he strongly recommends the writer's loading formula (M-Loading No. 3), as overcoming the objections heretofore advanced against equivalent uniform loads, and as yielding stresses which "provide for any of the critical engines of to-day, and, probably, by proportion, for those of years to come."

Mr. MacCornack also endorses the writer's paper, and recommends the proposed wheel-load diagram (M-Loading No. 1) as fairly representative of the actual loads in use. He would adopt such a system of wheel concentrations for examining existing structures and for designing bridge floors and short spans, and would use a loading formula equivalent to M-Loading No. 3 for designing all other bridge members. If preferred, he suggests, the load formula (M-Loading No. 3) might be modified to coincide more closely with the engine loading, and such a modification is submitted by the writer in Fig. 22. Mr. MacCornack rightly emphasizes the point that the important consideration is not the form of the loading specification, whether it is an engine diagram or an equivalent uniform load formula, or any modification

* See pp. 495-531.

of them, but that the loading adopted should represent correctly conditions in all parts of the bridge structure.

Mr. Disney endorses the writer's thesis. He regards all three of the alternative proposals as having distinctive advantages over the present methods, but considers M-Loading No. 3 as ideal, as "it is a formula which can be memorized and used, without tables or charts, to find the moments and shears at any point of a span by a simple slide-rule operation."

Mr. Disney, however, questions the applicability of the M-loading to Canadian railways on which, he states, the Santa Fé type is the present limit of locomotive development. In reply, the writer would direct attention to Plate XIV, which shows that the M-loading is better adapted than the Cooper loading to give economical protection to Santa Fé locomotives.

Mr. Ammann admits that unquestionably the proposed M-loadings have the advantage over the Cooper loading of greater simplicity. He points out, however, that the Cooper loading has been widely adopted as a standard, and that tables and diagrams are available to simplify the calculation of stresses for that loading. In reply, the writer submits that the mere fact of any standard being already in use is insufficient justification for retaining it when it is found to yield disproportionate and uneconomical results, and that the argument of tables and diagrams being available for applying the Cooper loading is cancelled by the publication of a complete set of similar tables and charts for the M-loading.

Mr. Ammann makes the statement that the proposed M-engine bears no greater similarity than the Cooper engine to any of the seven heaviest existing engines and those likely to be evolved in the future. The writer submits that this statement is refuted by a glance at the diagrams of the engines under consideration. The 5-axle grouping, which is the essential feature of the Santa Fé and Decapod engines, and the double set of drivers, which is the essential feature of the Mallet engines, are both included in the M-engine diagram, and neither of these features is represented in the Cooper engine diagram. What is more important, the M-loading is a more correct representation than the Cooper loading of the stress-producing effects of Santa Fé and Decapod engines, which represent the leading and most significant types of modern and prospective future engine development.

Mr. Ammann claims that the Cooper loading agrees as closely as the M-loading with the composite of the heaviest existing loadings; but, in his attempt to prove this, he does not take the Cooper loading as ordinarily specified and understood, but devises instead an arbitrary modification of the form E-72 + 6 000. This does not meet the issue, as this is not the Cooper standard under consideration, but a new loading devised to overcome one of the deficiencies of the Cooper standard. Fig. 6 shows the correct comparison, which clearly refutes Mr. Ammann's contention.

Mr. Ammann also claims that there are large discrepancies between the M-60 loading and individual heaviest existing engines. If such discrepancies exist, they are merely the inevitable variations between a maximum composite and its lesser components, variations which are on the safe side and which

provide a desired margin for future increase of stress at temporary low spots in stress-producing effects or in locomotive development; they are variations which will be reduced by future development toward the maximum composite instead of being (as in the case of the Cooper loading) variations which will become worse with the progressive departure of locomotive design from an outgrown standard.

Moreover, as shown by Plate XIV, for individual engines of the dominant types, the discrepancies of the M-loading are much smaller than those of the Cooper loading.

The suggestion of correcting the disproportion in the Cooper loading by changing the ratio of train weight to locomotive weight, made by Mr. Ammann, has already been answered. The proposed modification will upset existing ratings and lead to errors, and will only partly solve the problems under consideration. It will not correct the disproportion of stress within a given span, nor the disparity of ratings between different short spans, and it will not obviate the need of revising existing stress tables and ratings. If any change is to be made, no half-way measure should be used in correcting the standards; a fresh start, based on the latest developments, will prove more satisfactory in the long run than attempts to tinker with an outgrown standard.

Mr. Ammann suggests the possibility that the weights of locomotives and cars may continue to increase at different rates. As it is impossible to foretell future relative development, the writer submits that the present-day ratio of driver weights to car weights constitutes the best basis available for establishing a new loading specification. It is unquestionably a more logical basis than the outgrown ratio which obtained a generation or more ago. Mr. Ammann's suggestion of a flexible specification in which the individual engineer may arbitrarily vary the ratio of the uniform train load to the engine concentrations would only lead to chaos in standards, ratings, and stress tables.

Mr. Seaman endorses the writer's paper and conclusions and predicts that in the future bridges will be designed for the M-loading. He mentions the conversion tables which will facilitate the altering of recorded ratings of the railroads; with this object in view, the writer has added the conversion tables (Tables 11 and 12), and the conversion charts (Figs. 23 and 24) to this discussion, and has prepared the conversion table (Table 17) published in the report of the Special Committee on Specifications for Bridge Design and Construction.*

Mr. Mead also endorses the writer's paper. He declares that the necessity for revision is apparent, and points out that the present loading system gives unbalanced designs.

Mr. Mead strongly endorses the writer's conclusion that train loads and locomotive loads have not increased in the same ratio. The contention that the writer assumed an insufficient train load behind the modern locomotives considered in this paper, is answered by the following quotation from Mr. Mead's discussion:

* See p. 516.

"At present, train loads do not exceed 6 000 lb. per lin. ft. of single track, except in a few instances which will not affect the conclusions reached. Furthermore, the present design of cars must be changed so radically, in order to increase this train loading, that it seems as if this uniform load would represent a maximum train loading for an indefinite period."

Mr. Mead expresses a preference for M-Loading No. 2 (the uniform load plus spot loads) for its simplicity, flexibility, and ease of application.

Mr. Keith endorses the writer's paper and adds an interesting review of some early loading specifications. A useful lesson may be derived from Mr. Keith's account of a specification adopted by the New England Railroad Company in 1882, consisting of a live load headed by a Consolidation locomotive—a new type at that time—with four driving axles of 24 000 lb. per axle. This specification was criticized and ridiculed, many railroad men declaring that such a heavy engine would never be built. "Within three years that road had engines in use heavier than those that had been considered as excessive." History repeats itself; and engineers continue to make the mistake of under-rating future development.

Mr. Lang disagrees with the writer's conclusions with an animus which, under the circumstances, it is difficult to understand. The writer has simply given his best efforts to the analysis and study of a problem which appears to be of importance and of general interest, and it would be regrettable if such investigations are to be taken out of the field of sincere professional discussion into the domain of factional controversy.

If, as Mr. Lang states, the paper gives the impression of being "primarily an attack on the Cooper system of loading", that was far from being the writer's intention. The writer joins with Mr. Lang and others in a sincere admiration of Mr. Cooper's contribution and in a full appreciation of its past value in bridge design; but, as has been pointed out by others as well as the writer, the Cooper loading diagram has outlived its usefulness. It has served its purpose well, but new conditions have arisen as a result of locomotive developments far exceeding Mr. Cooper's anticipations, and surely engineers may make revisions to suit these new conditions without detracting credit from the pioneer who blazed the trail. The writer's position in this matter is that expressed by the dean of railroad bridge engineers, Mr. Bland, who has stated that, "if Mr. Cooper was still alive, he would share this view and be among the first to devise a new method and one which not only conformed to existing engines and train loads, but looked forward and beyond."

The principal criticisms made by Mr. Lang have been answered at length. The selections of engines and following train loads in the writer's investigation have been fully explained and defended; they have received the confirmation and endorsement of many eminent bridge engineers as the correct and proper basis for a modern loading specification.

Mr. Lang's question why the special engines used by the writer should be followed by a train load of only 6 000 lb. per lin. ft., is answered by the simple fact that that is the maximum weight of train load actually used behind the engines under discussion. In fact, the heaviest cars to be found behind these

locomotives are generally 210 000-lb. coal cars, which give a maximum train weight (with all cars carrying full overload) of only 4 770 lb. per lin. ft.

If these modern engines and their actual following train loads are compared with the E-40 loading, confirmation will be found of the statement challenged by Mr. Lang, that "the axle concentrations have increased 100%, whereas the weight of train per linear foot has increased barely 50 per cent." This disproportion is still further accentuated when the Pennsylvania H-C-1-S engine with axle loads of 86 400 lb. is considered.

Mr. Lang contends that the Cooper E-73 with a train load of 7 300 lb. per lin. ft. "more nearly approximates present-day equipment and rolling stock." With the exception of some special ore cars, trains will not be found exceeding 6 000 lb. per lin. ft. As a rule, the maximum train weight is considerably less than this figure, even behind the engines which Mr. Lang characterizes as "extreme".

Mr. Lang criticizes the M-loading as perpetuating a material difference in the effect of the engine and train load on bridge structures. The same criticism also applies to the E-loading; the essential distinction is that the Cooper loading would perpetuate the ratio that obtained more than thirty years ago, whereas the M-loading makes a fresh start with the ratio that obtains to-day. If there was any practical way, without compensating disadvantage, to make the specification flexible in this respect, the writer would welcome it; in the absence of such expedient, it is surely preferable to base a specification on the present ratio than to perpetuate a ratio that has long been outgrown.

Mr. Lang questions the writer's suggestion that design loads should be 25 to 50% heavier than the average present loading. The writer merely made this suggestion in the light of repeated past experience with the obsolescence of bridge structures under unanticipated load increase. This subject is involved with the question of proper unit stresses. Where a unit stress of 20 000 or 24 000 lb. per sq. in. would be justified, the current practice is to use only 16 000 lb. per sq. in., in order to provide for a possible load increase of 25 or 50 per cent. The writer maintains that it would be more rational and would give better designs to increase the assumed live loads instead of reducing the unit stresses. Designing with ultimate unit stresses for ultimate anticipated loads will give the best proportioned structures, with the metal distributed where it will do the most good in insuring their longevity. When this fundamental principle receives recognition in specifications in the form of increased unit stresses, the design load can properly be augmented from M-50 to M-60 or M-75.

Mr. Lang submits a table of E-ratings for M-60 (Table 10), in which he shows apparent errors in the writer's figures by comparison with his own. The values which Mr. Lang gives are taken from Table 2, which refers to the composite loading; whereas, he should have calculated his values from Table 3, which refers to the proposed typical engine loading (M-Loading No. 1). In addition, Mr. Lang has included in his figures the alternative loading of two axle concentrations which are not a part of the Cooper loading, but have

been added to it (with variations) to correct some of the deficiencies of that loading at short spans. (In the proposed M-loading, the necessity for such alternative concentrations is obviated.) With these corrections, the apparent errors in the writer's values will be found to vanish.

Mr. Lang attempts to show that the proposed M-loading does not vary appreciably from the Cooper system of loading, but his own values in Table 10 are a sufficient refutation, as they show a variation of 27% in the E-rating for M-60, from a maximum of E-77.3 at a 40-ft. span to a minimum of E-61.0 at a 300-ft. span. A variation of 27% is not one to be ignored. With the modified M-engine diagram adopted by the Special Committee on Specifications for Bridge Design and Construction, the variation from the Cooper loading is still greater, amounting to 38 per cent. (See Table 17*.)

Mr. Lang also attempts to prove that the Cooper loading fits the composite of the heaviest locomotives better than the proposed M-loading. Inspection of Fig. 4 and Fig. 6 will prove the contrary. Fig. 4 shows that with the exception of a few isolated points, the M-loading agrees with the composite within 1%, for all points of all spans up to 1 000 ft.; the few larger variations found are confined to very short spans, with extreme values of — 4 and + 6 per cent. Fig. 6, as well as Plate IX, shows that the Cooper loading varies from the composite by about 25 per cent. If E-60 is used, this variation from the composite is found at short spans; if E-73 or E-75 is used, the 25% variation is found at long spans. Mr. Lang uses E-73, but omits the long spans from his diagram (Fig. 17), so that the 25% variation from the composite is not revealed.

With the modification of the M-loading shown in Fig. 19, the variation of M-60 from the composite has been augmented at short spans, on the side of safety; but even with this increase, M-60 shows less variation from the composite than E-73. The maximum variation of center moments from the composite is 21.3% for E-73 and only 11.9% for M-60, and the maximum variation of end shears from the composite is 18.3% for E-73 and only 9.3% for M-60. Even this comparison is not fair to the M-loading, as these variations of M-60 from the composite are confined to the shortest spans and represent an excess intentionally introduced for the better protection of these short spans in which stress effects are most likely to be increased by future developments.

Mr. Lang emphasizes the point that it would involve too much work to re-classify the 250 000 railroad bridges in the United States. This would not be necessary. Moreover, it will be found that very few railroads have made much headway in "Cooperizing" their lines; the bridges for which E-ratings have been calculated and recorded constitute a small proportion of the total. For the bridges as yet unclassified, it will be as easy to fix the M-ratings as the E-ratings; and for the others, even if deemed necessary, it requires very little work per structure to translate E-rating to M-rating by means of a conversion table or chart (Table 17,* as presented by the Special Committee on Specifications for Bridge Design and Construction, mentioned previously,

* See p. 516.

Tables 11 and 12, and Figs. 22 and 23). This conversion to M-rating will soon be justified in the time saved in judging the capacity of the structure for heavy loadings.

Mr. Lang makes a special point of showing that about 96% of the railroad bridges are less than 120-ft. span, and that at less than that span the differences between the E-loading and the M-loading are unimportant. The spans exceeding 120 ft. may be only 4% of the total number, but they are considerably more important than that small percentage would indicate. A single long span may outweigh, in importance and in cost, a hundred small viaduct spans in its approaches. One would not think of limiting the specifications to 120 ft., and letting all longer spans take care of themselves. Engineers are not writing specifications limited to plate-girder spans. Moreover, a glance at Plate XIV, or Table 17* of the report of the Special Committee on Specifications for Bridge Design and Construction, reveals a considerable difference between the M-loading and the E-loading even in short spans; Table 17* shows that M-60 is equivalent to E-83 at a 35-ft. span and to E-69.8 at a 98-ft. span, or a variation of 19% below a 100-ft. span.

Mr. Welty is one of the authors of the Sub-Committee report† which influenced the American Railway Engineering Association to retain the Cooper loading in 1920; and, in his discussion of the writer's paper, he maintains his preference for the Cooper loading "with possibly a modification in the uniform load following the series of wheel loads."

Mr. Welty's argument that most railroad bridges have span lengths of less than 100 ft. and that, for such spans, it makes little difference what loading is used, has already been answered.

For longer spans, Mr. Welty points out that the difference between the loadings can be corrected by changing the Cooper loading, using E-73 + 6 000 instead of the Cooper standard, E-73 + 7 300. This suggestion has already been discussed at length, and it has been shown that this would not be a satisfactory solution of the problem. Plate XIV shows one of the reasons. It would not be as good a solution as a change to M-60.

Professor Williams endorses the writer's investigation as promoting economy, adequacy, and consistency in bridge design. He declares that, in a sense, the Cooper loadings have been a deteriment to structural engineering, and that lack of balance in most designs resulting from the application of the E-rating to express modern loadings even exceeds the 37% given by the writer.

Of the three forms of loading specification proposed, Professor Williams expresses a preference for the typical engine diagram (M-Loading No. 1), as he considers equivalent uniform loads insufficiently accurate to represent the actual effects of wheel concentrations. This point, however, is open to considerable difference of opinion.

Professor Williams emphasizes a point which the writer has made elsewhere, namely, that the problems of loads and stresses are interrelated. The present working stress of 16 000 lb. per sq. in. is too low. To provide properly for

* See p. 516.

† *Bulletin*, Am. Ry. Eng. Assoc., January, 1920, p. 511.

future increase, the factor of safety should be inserted in the loadings rather than in the unit stresses. The correctness of the principle of "ultimate loads with ultimate unit stresses" cannot be denied, and the Profession will have to come to its acceptance. This thought has been the basis of all the writer's studies for his paper, and may help to explain why his recommendations of design loadings are higher than those customarily regarded as adequate.

Mr. Jones weighs the difficulties involved in arriving at a uniform loading specification of enduring application. He endorses the indictment of the Cooper system because of the discrepancies between it and present-day developments, and indicates the reasons for the use by so many railroad companies of loadings other than the Cooper loadings. Although a new specification derived on essentially the same assumptions as those of Cooper may prove to be subject to the same local limitations in the light of future developments, Mr. Jones agrees in the desirability of making a fresh start and of having a uniform loading specification throughout the United States. He expresses the hope that, "even though the permanency of a new standard adopted to-day may be voted dubious, one may be adopted, if only as a second step toward the ultimate."

Mr. Jones prefers the typical engine diagram (M-Loading No. 1) as an outline which can be easily reviewed as mechanical developments are made. He, therefore, chooses it for publication in the specifications; he points out, however, that the designer will probably forget the wheel-load diagram, paste the table (Fig. 20), or chart of equivalent uniform loads (Plate XIII), in his handbook, and, perhaps, memorize the equivalent loading formula (Equation (4) or Equation (12) for emergencies, and thereby save precious hours of computation.

Mr. Woodruff endorses the writer's argument that the Cooper loading does not fit present conditions, and emphasizes the fact that the divergence is likely to be increased by future developments.

Commenting on the proposed M-loading, Mr. Woodruff raises some questions which have already been discussed. His objection on the ground that some of the locomotives considered by the writer are in limited use, is answered by the argument that these locomotives represent the maximum of each of the modern types and were considered merely as the best indication of the trend of future development. He suggests that the M-loading does not provide adequately for short spans (although it does so more generously than the Cooper loading). The writer has anticipated this suggestion by changing the spacing of the heavy drivers in the M-engine diagram from 5½ ft. to 5 ft.

Mr. Woodruff suggests an increase of 10% in the uniform load of the proposed M-loading on the ground that in the future the train load may increase more rapidly than the engine load; but the writer, guided by past experience and supported by others in these discussions, does not agree with this prediction. The economics of railroad operation demand continued development and augmentation of locomotive weights, but not of car weights. The question of relative train load affects only the long spans; and for those spans the lower average weight (per linear foot) of long trains should be taken into account.

Mr. Woodruff declares his preference for M-Loading No. 1, as it facilitates judgment of heavy shipments.

He suggests designing directly for M-75, both for simplicity and economy; but this is exactly what the writer meant when he recommended designing for the M-60 loading with a provision for 25% overload; he expressed it in that form in order to keep before the designing engineer the intent of the 25% margin, and the thought that the unit stress may then be justifiably increased from 16 000 to 20 000 lb. per sq. in.

Mr. Thomson endorses the writer's paper. He points out that there is more uniformity now than in the past in the loadings used by different railways, and emphasizes the importance and value of proper standardization in this field.

Professor Young characterizes the data presented by the writer as "so convincing as to leave no other course open than to revise the present obsolete locomotive loadings." He adds: "The author has established that, for heavy loadings on first-class railway systems, the Cooper system is inadequate and that the new proposed system would give safe and logical results." Professor Young expresses the hope that the American Railway Engineering Association will reconsider its decision to abide by the Cooper system.

Although Professor Young endorses the M-loadings as "admirably suited to the heavier lines operating with steam locomotives," he questions whether they are as closely applicable to average or second-class lines. Plate XIV shows that any line on which Santa Fé engines are operated will be better protected by the M-loading than by the E-loading. Mr. Bland, of the Pennsylvania System, has found, from independent studies, that the M-loading is also admirably adapted to protect Consolidation and Mikado locomotives. The M-equivalents found by him are given on page 710. Accordingly, for average or second-class lines, even those on which a Mallet locomotive will never be used, it will be found to be safe and economical to adopt the M-loadings.

Professor Young also mentions the increasing importance of the electric locomotive. Although this type of traction has not developed sufficiently to permit judgment as to future standards, the writer is prepared to state that the more advanced designs of the electric locomotive show features which fully justify the adoption of the M-loading. The new Norfolk and Western type of electric locomotive has eight axle loads of 70 250 lb. each, in two closely spaced groups, and, therefore, resembles a Mallet wheel diagram. The writer has calculated its stresses and finds that it is safely and economically represented by M-52.

Of the alternative forms of the proposed loading, Professor Young favors the loading formula (M-Loading No. 3) as most direct and scientific, and therein agrees with the preference of the writer. It does not seem expedient at this time, however, to introduce such an innovation into general specifications.

Mr. Godfrey endorses the writer's conclusion that, for the heaviest locomotives and train loads, the Cooper loading does not give consistent stresses nor economic designs. This, however, Mr. Godfrey does not regard as a

sufficient reason for making a change. In the writer's opinion, Mr. Godfrey's arguments involve the fallacy of giving undue prominence to the existing ratings of old bridges, and of under-rating the importance of the proper design of new bridges. Sooner or later, all capacity ratings will have to be reviewed or revised for loading development, and, eventually, all bridges will have to be rebuilt. It appears, therefore, to be a short-sighted policy to perpetuate an old loading system which, it is admitted, does not correspond with the latest in modern engine construction, in order to save the temporary (and over-estimated) trouble of making a change. Moreover, even for the rating of bridges, the M-system will be found to be a time saver, for it will facilitate comparison of rated capacities with actual modern loadings.

Apparently forgetting his own arguments against a change of loading standard, Mr. Godfrey proposes to modify the Cooper loading in order to correct its shortcomings. This proposal would not obviate the need of revising existing bridge ratings, the difficulty of which Mr. Godfrey has cited as his principal objection to a change of specification.

The two modifications of the Cooper loading proposed by Mr. Godfrey are a reduction of the uniform train load behind the locomotive, and an increase in weight of the alternative axle concentrations; the proposed M-loading combines these features in a much simpler diagram, and gives the desired results without the use of any alternative loads.

Mr. Robinson expresses a strong preference for the Cooper loading. He cites, as his authority, the report of the Sub-Committee of the American Railway Engineering Association in preparing the Specifications of 1920.* The writer, in his investigation, used the same locomotives that were considered in that report, and from them derived the proposed M-loadings. It appears inconsistent to cite that report as an authority and, at the same time, to criticize the writer's loading assumptions.

Mr. Robinson contends that not many Mallet locomotives will be built in the future, because the Santa Fé and Mikado types are more efficient. According to Mr. Bland's findings, the M-loading fits the Mikado stresses consistently, and, as shown by Plate XIV, the M-loading fits the stress-producing effects of locomotives of the Santa Fé type much better than the Cooper loading. Mr. Robinson admits that it may prove necessary later to revise the Cooper loading; but, if a change is anticipated, there is no time like the present for making it.

Mr. Robinson also argues that M-60 is too heavy for locomotives in general use. If that is true, the railway engineer can specify M-50 instead. The essential point is not the index number, but the adoption of a correct type of loading to produce properly balanced designs for anticipated development.

Mr. Moore endorses the writer's investigations and conclusions. He makes the important point that any change which better meets the tendencies of locomotive design is an advance.

He declares his preference for M-Loading No. 1 (the typical engine diagram), as it permits stresses to be checked by judgment. The requirements

* *Bulletin*, Am. Ry. Eng. Assoc., January, 1920, p. 511.

of the lighter lines, he points out, can be met by reducing M-60 to a lower index.

Mr. Moore also raises the interesting question of electric locomotives. He mentions the Norfolk and Western freight locomotive that has four axles of 70 250 lb. each, concentrated in a distance of 16 ft. 6 in., with another similar series of loads 22 ft. away.* The writer agrees with Mr. Moore that the stress-producing effects of this locomotive are nearly as great as those of the series of steam locomotives used in deriving the composite. Calculations show that this locomotive, followed by 210 000-lb. cars, is closely and consistently represented by the M-52 loading. (In the Cooper system, the equivalent is E-67 at short spans and E-50 at long spans, a variation of 34 per cent.)

Mr. Moore believes that increase in length of locomotive with new wheel arrangements is more probable in future developments than further intensification of axle concentrations. That would be another argument for the M-loading, as it provides for more axle loads per locomotive than the Cooper loading.

Mr. Moore makes one suggestion open to question, namely, that the uniform train load behind the locomotive should be kept constant. There are practical obstacles to such a specification. The writer has chosen a middle ground between this suggestion to keep the train load constant and the contentions of those who believe that future train loads will increase more rapidly than locomotive loads. Economic considerations would indicate that Mr. Moore's prediction is the more likely. The middle course adopted by the writer follows the line indicated by past experience, and is dictated by practical expediency.

Mr. Moore concludes that the proposed loadings are progressive and will be justified by future locomotive developments; and he expresses his conviction that the M-loading will soon be generally adopted.

Professor Wing devotes his discussion principally to a consideration of the choice of form of loading specification. He prefers the actual heavy engine diagrams for rating the capacity of existing bridges, and the typical engine diagram (M-Loading No. 1) for the design of new bridges. For the rating of existing bridges, he will find that it will save time to determine and use the M-equivalent of the governing locomotives on any line, similar to the series of M-equivalents established by Mr. Bland for the different governing locomotives on the Pennsylvania Lines.

Mr. Stewart regards the writer's proposals as extreme, because they are based on maximum instead of average locomotives. This point has already been discussed at length, and the writer's position that a progressive loading system should anticipate future development really agrees with Mr. Stewart's admission that:

"As the tendency has been * * * for the weight of rolling stock to increase slowly, the standard loading should tend to be more nearly normal to the heavier rather than to the lighter existing or probable locomotives."

The writer does not agree with Mr. Stewart's statement that the M-loading cannot be reduced to fit the lighter classes of locomotives. Independent

* *Engineering News-Record*, September 14, 1922, p. 437.

calculations made by the writer and by Mr. Bland, of the Pennsylvania System, show that the M-system furnishes consistent ratings even for Consolidation locomotives. The M-loading is as good as the Cooper system (and simpler) for the lighter locomotive types, and it is better than the Cooper system for the heavier types.

Professor Jacoby endorses the writer's comparisons between the composite of the heavy modern loadings and the Cooper standard now in use. He points out that it is an easy matter for any railroad to make its own comparison of its governing locomotives with the M-standard, and to determine how closely a reduction of index from M-60 would fit the results. He recommends the M-Loading No. 1 (the typical engine diagram) as the most expedient and practical.

Mr. Fowler endorses the writer's position that the Cooper system is badly out of date and out of time with modern bridge engineering, and that a composite loading is the correct solution of the problem. He also agrees with the writer that the Cooper standard perpetuates the incorrect ratio of train load to locomotive load, and cites, as an instance, the fact that the weight of the train behind the heaviest locomotive on the Michigan Central Railroad is only 4 000 lb. per lin. ft.

Mr. Loweth declares himself in sympathy with the movement to simplify and make more practical the typical engine and train loadings used for proportioning railroad bridges. He cites the repeated past experience of designing bridges for the heaviest locomotives known with a margin for possible increase, only to find the assumed loading exceeded by heavier locomotives before the bridge was completed. Surely, this is a sufficient answer to those who criticize the proposed M-loading as being based on locomotives heavier than the average.

Mr. Loweth's statement that electric locomotives produce less impact is a consideration which, in the writer's opinion, should be covered in the impact formula and not in the loading specification. He suggests further that electric locomotives will be less frequently double-headed than steam locomotives, which is an argument in favor of the M-engine diagram, as that diagram consists of a single-headed locomotive.

Mr. Modjeski endorses the writer's conclusion that the Cooper standard loading does not properly represent modern loading conditions. He also suggests modifying the Cooper loading and using E-73 + 6 000 instead of M-60, which suggestion has already been discussed.

Mr. Modjeski indicates a perference for the second alternative, namely, a uniform load with floating concentrations (M-Loading No. 2).

Mr. Turner presents a theoretical analysis of impact, particularly as affected by speed. From this, he concludes that the impact for E-60 loading at passenger train speed would be nearly equal to that of an E-75 loading at freight train speed. Similar relations can be established, of course, between different indices of the M-system of loads, but the writer does not see how they affect the choice as between the Class-E and the Class-M loadings. The proper place to cover considerations of impact is in the specification for

impact, not in the specification for loads. If it is Mr. Turner's idea that a loading specification should be based on passenger locomotives and not on freight locomotives, the writer does not believe that the Profession would share such a view.

In closing, the writer wishes to state that, after reviewing the discussions, he is confirmed in his conclusions that the Cooper engine diagram does not properly represent modern loading conditions; and he submits the M-loading (Fig. 19) as the final result of his studies to determine the best possible loading specification to meet present-day and future loading conditions.

The writer wishes to take this opportunity of recording his gratitude to all those who have helped in connection with this paper: to O. E. Hovey, H. B. Seaman, J. C. Bland, Members, Am. Soc. C. E., and Mr. F. A. Howard, for their helpful encouragement; to H. T. Welty, F. E. Schmitt, C. E. Fowler, and W. G. Grove, Members, Am. Soc. C. E., for valuable suggestions received; to Messrs. G. Johnson and J. London for loyal and generous assistance in preparing the tables and diagrams; to the Secretary of the Society and his staff for their helpful co-operation; and, above all, to those who have so generously enriched the paper by contributing their discussions.

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Paper No. 1521

THE WATER POWER PROBLEM*

A SYMPOSIUM

By Messrs. Arthur P. Davis, John P. Hogan, O. C. Merrill, F. W. Peek, Jr., Charles D. Marx, Gerard H. Matthes, John D. Galloway, Harris J. Ryan, Harry W. Dennis and H. A. Barre, C. F. Loweth, and Frederick Hall Fowler.

With Discussion by Messrs. J. M. Howells, M. M. O'Shaughnessy, J. B. Lippincott, C. S. Jarvis, D. C. Henny, F. H. Tibbetts, Louis C. Hill, C. H. Birdseye, Edward H. Sargent, William T. Lyle, Howard R. Farnsworth, J. W. Swaren, J. P. Jollyman, F. P. Williams, G. B. Herington, W. G. Hoyt, Gerard H. Matthes, and Frederick Hall Fowler.

* Presented at the meeting of the Society at San Francisco, Calif., on October 4 and 5, 1922.

THE COLORADO RIVER DEVELOPMENT

By Arthur P. Davis,* Past-President, Am. Soc. C. E.

The Colorado River and its tributaries constitute one of the greatest potential assets of the United States. Already serving more than 2 000 000 acres of farm land with irrigation water, it will eventually serve treble that area. It drains parts of seven States, and flows for more than 100 miles on Mexican soil. Before the building of railroads, it was navigated for more than 400 miles, and although now mainly abandoned by navigators, it is still navigable. This, and its interstate and international character, makes its problems peculiarly national.

Its great power possibilities are one of its most remarkable characteristics. Rising on the summit of the Rocky Mountains, at altitudes exceeding 14 000 ft., it flows rapidly through canyons abounding with rapids, to nearly sea level. It drains an area of about 244 000 sq. miles, including large areas of heavy snow fall, and its annual average discharge is about 18 000 000 acre-ft. This large volume of water, with its immense fall, indicates a total of 8 000 000 h. p., of which three-fourths is susceptible of development whenever reasonable markets will justify it. In addition, its tributaries, widely distributed and already partly developed, have vast possibilities of power. Fig. 1 is a map of the Colorado River Basin showing Government irrigation projects, constructed and proposed, power plants in operation, possible power sites, etc.

Its upper courses are mostly mountainous, and traverse a few valleys of moderate area. Its middle reaches are through plateaus scored by profound canyons, constituting, topographically, one of the roughest regions of the world. the lower stretches pass through hot, dry valleys of alluvial silt, which are rich in all kinds of plant food and which have a growing season of twelve months. If properly regulated, the Colorado is ample to supply the much needed water for the irrigation of these semi-tropical valleys and to make them produce a wonderful abundance and sustain a dense and prosperous population. The discharge of the river is sometimes less than 5 000 cu. ft. per sec. and the mean flow is about 24 000 sec-ft. The peaks of its floods at Laguna Dam are at times more than 200 000 cu. ft. per sec. The low-water flow of the river is already appropriated for irrigation and cannot be greatly extended without storage.

The complete and efficient use of the river for power also requires its regulation by large storage works. The most important and the most urgent need of storage, however, is in the control of the destructive floods which annually pour over the lower valleys. Every large alluvial valley has its problems of river control, but those of the Colorado are unique, on account of the peculiar topographic conditions on the lower river.

The Gulf of California formerly extended nearly 150 miles farther north than its present head, and the Colorado River emptied into its eastern margin about 100 miles south of the head, carrying annually into it more than 80 000 acre-ft. of silt. This deposition built a great alluvial delta which finally

* Washington, D. C.

extended across the Gulf and cut off the connection of the northern end from the sea, converting it into an inland basin from which the water has been



FIG. 1.

mostly evaporated, leaving in its bottom the Salton Sea covering 300 sq. miles, with a surface nearly 250 ft. below sea level.

Like other similar streams, the river, laden with sediment, continually builds up its channel and immediate banks, and forms a ridge growing steadily higher, until the stream becomes so unstable that, in some great freshet, it abandons this high channel and follows some course on lower ground. In this manner, the stream swings back and forth over its delta which has been built to a height of 30 ft. above sea level, where it forms a barrier between the Salton Sea and the Gulf of California. The direct distance from this ridge to the Gulf is about 70 miles, and to the margin of the Salton Sea, the distance is a little more. As the inland sea is about 250 ft. below the Gulf, there is a constant tendency to flow northward, and if the river should break into this basin in flood time, it would rapidly erode a deep channel and would be exceedingly difficult to stop. Unless diverted, the river would fill the Salton Sea to sea level, or higher, and submerge the Imperial Valley, destroying all the towns, railroads, and farms, as there is no escape for the waters of the Salton Sea except by evaporation. This menace would be largely removed by the reduction of the floods of the Colorado River, and the control of its waters.

Some engineers have held that the sediment carried by the Colorado is so voluminous that the construction of storage reservoirs in the lower basin should not be attempted, but that such reservoirs should be built only in the upper basin, where the waters are comparatively clear. In the upper country, numerous good storage sites have been surveyed and found to be feasible. They intercept the major part of the water supply, but their use for the purposes of the lower basin is subject to serious objections.

If all the reservoirs found to be feasible above the canyon regions were constructed, more than one-half the drainage area would still be unregulated. Although this is the more arid part of the basin, it furnishes occasional serious floods, and the flood problem could not thus be solved. The Gila River alone, although draining only 21% of the basin, and contributing only 6% of the water, occasionally discharges floods the peak flows of which are more than those of the Colorado at Laguna Dam. These floods, although of short duration, and of relatively small total volume, are a menace and require separate storage. This is true also of any other large area not regulated by the reservoirs suggested. Another serious objection to so using the reservoirs of the upper basin, is that it would destroy their usefulness for local purposes in the upper States and thus virtually destroy the large resources dependent on them in their vicinity.

Fortunately, it has been disclosed that it is feasible to construct a dam in the granite gorge of Boulder Canyon, that will intercept practically all the drainage above the Gila and form a reservoir of any desired capacity, sufficient to regulate the entire flow of the river to any desired regimen of discharge. Fig. 2 is a view of the dam site in Boulder Canyon. Such a reservoir would not only eliminate the flood menace of the river, but would also furnish the head necessary to develop 600 000 firm h. p., and this same regulation would meet the requirements of irrigation in Arizona and California. The regulated flow would also supply two large power sites below it, still nearer the present markets. It is also fortunate that the Boulder Canyon site is located so as to be within transmission distance of large available power markets in

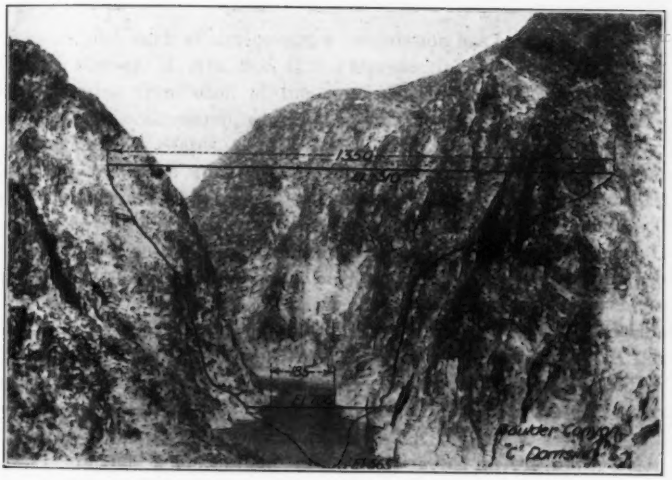


FIG. 2.—VIEW OF BOULDER CANYON DAM SITE.

Arizona, California, Nevada, and Utah. It is within about 30 miles of a transcontinental railway, an important consideration as regards accessibility and cost.

The sediment delivered by the Lower Colorado River is estimated to be about 90 000 acre-ft. annually. This quantity can be stored by providing capacity in addition to that required for other purposes; a capacity of 9 000 000 acre-ft. would store the silt for a century, and larger capacity can be provided if desired. Eventually, it will be necessary to provide regulation above, but this is so far in the future that the upper reservoirs will doubtless be built in ample time. Even if the Boulder Canyon Reservoir should entirely fill with sediment, the power head will be unimpaired. Thus, Nature has provided means for removing any conflict of interests between the upper and lower basins of the Colorado, and the way is open for an amicable agreement on a division of its waters, which are ample for all feasible uses in all the States concerned.

To accomplish such an agreement, a commission has been appointed, headed by Herbert Hoover, M. Am. Soc. C. E., representing the Federal Government. A representative from each of the seven States directly interested is also included on this commission, which, it is hoped, will soon reach an agreement acceptable to all the States.

This will leave the upper States free to develop all their irrigation and power resources without the menace of antagonistic claims in the lower States. Feasible reservoir sites have been surveyed at Flaming Gorge on the Green River, at Juniper on the Yampa, and at Dewey and Gore Canyon on the Grand River. A site also exists at Ouray, on the Green River, which is capable of development to a capacity of more than 10 000 000 acre-ft., forming the largest reservoir in the upper basin, but it would occupy a valley that is desired for use as a railroad route between Denver, Colo., and Salt Lake City, Utah, and perhaps may be devoted to that use. Numerous reservoirs of less capacity on the smaller tributaries of the Colorado are available for local use in irrigation, with perhaps some power development.

About 1 526 000 acres of land are now irrigated in the upper basin and there is a possibility of bringing this development to more than 4 000 000 acres, or nearly three times the present area.

The lower basin, exclusive of the Gila Basin, now includes about 510 000 acres of irrigated lands, and this area can be increased to about 1 250 000 acres, exclusive of Mexico. Fortunately, the supply of water, if wisely regulated and applied, is ample to carry out all these projects and to develop more than 6 000 000 h. p., not only without injury to irrigation, but to the great benefit thereof. It is important that the facts be fully ascertained and used in formulating a comprehensive plan for the best use of the water, and that the development follows this plan.

PRESENT TENDENCIES OF WATER POWER DEVELOPMENT IN NEW YORK STATE

By John P. Hogan,* M. Am. Soc. C. E.

The State of New York has an area of about 50 000 sq. miles, or approximately 2% of the area of Continental United States, excluding Alaska. It contains, however, one-tenth of the population and about one-eighth of the industries of the entire nation. Of an estimated central-station capacity of about 23 000 000 h. p. in the United States, more than 3 000 000 h. p., or about 13%, is in New York State, and the production there of about 6 000 000 000 kw-hr. is about 15% of the total for the entire country. It is estimated that the installed capacity in prime movers in the State is about 5 000 000 h. p., of which 1 300 000 h. p. is in the form of water power and 1 200 000 h. p. is in individual steam plants, leaving the steam capacity in central stations and electric railways about 2 500 000 h. p. The total undeveloped water power in the State, including one-half of the potential energy of Niagara and one-half of that part of the St. Lawrence River bordering on New York State, is about 4 000 000 continuous h. p., divided as follows:

Niagara River	2 000 000 h. p.
St. Lawrence River	800 000 h. p.
Delaware River	150 000 h. p.
Interior streams	1 050 000 h. p.

It is improbable that public opinion will permit complete utilization of the potential energy of Niagara, but competent authorities have estimated that at least 500 000 h. p. additional could be developed on the American side without detriment to the scenic effects of the Falls. Hugh L. Cooper, M. Am. Soc. C. E., considers the power on the St. Lawrence, either with or without a canal, as immediately available. Of the interior possibilities of 1 050 000 h. p., the speaker estimates that about 800 000 continuous h. p. are susceptible of economic development, and that an installation of about 1 600 000 h. p. would be justified to permit operation on the market load factor. Therefore, sufficient undeveloped water power is available to permit replacing all the present steam power in central stations and electric railways, provided the water power is of sufficient flexibility to meet market conditions. The present annual coal consumption in New York State for manufacturing, lighting, and transportation purposes, exclusive of heating, is about 37 000 000 tons, of which, it is estimated, more than one-third could be saved by the development of water power. With the normal average price of coal at about $7 per ton, it is estimated that capital expenditures of about $1 000 000 000 for this purpose would be justified.

General

Two general classes of water powers exist in New York State, as follows:

First.—The great continuous powers on the Niagara and St. Lawrence Rivers.

* Cons. Engr., New York City.

Second.—The irregular powers on the interior streams which are susceptible of regulation by storage.

The general characteristics of the present and proposed developments at Niagara and on the St. Lawrence River are too well known to the members of the Society to warrant their discussion in a paper of this character, except in as much as their operating characteristics effect the development of the irregular water powers and of steam auxiliaries. This paper, therefore, will be devoted largely to a description of the present developments on the interior or intermittent streams and the tendencies in power developments on them.

The striking physical features of the State of New York are the great Adirondack Plateau in the northeastern corner, rising to a maximum height of 5 300 ft. above sea level, and the Western Plateau extending through the central part of the State, from the Hudson on the east to the western boundary, at an average elevation of about 1 000 ft. above sea level. The Adirondack Plateau drains to the west, north, and east, into the St. Lawrence Basin, and to the south into the Atlantic Ocean through the Hudson and its tributary, the Mohawk. The Western Plateau forms the divide between the St. Lawrence Basin on the north and the Mississippi (Allegheny), Susquehanna, and Delaware Basins on the south.

Fig. 3 is an isohyetal map of the State and Fig. 4 is a map with lines of equal run-off. These maps are subject to the errors due to an insufficient number of stations and lengths of records, but they give a general idea of precipitation and run-off conditions.

It will be noted that the maximum average annual precipitation is on the Adirondack Plateau (particularly in the southern and southwestern part) and along the valley of the Hudson, whereas, in general, the precipitation on the Western Plateau is low. Thus, the maximum annual precipitation on the Adirondacks is in excess of 60 in., and the average is about 44 in., as compared with an average of 39 in. for the entire State; whereas, on the Western Plateau, the average annual precipitation varies between 28 and 48 in., with an average for the entire area of about 35 in. The disparity between the run-off of the respective regions is still more striking, the average annual run-off for the Adirondack Plateau being about 26 in., whereas that for the Western Plateau is about 16 in. When distribution throughout the year, or variations through a series of years, is considered, the advantage of the Adirondack Plateau is still greater. It has greater lake area, more natural storage basins, and almost no cultivated ground, whereas the Western Plateau contains the most fertile land of the State and is in a high state of cultivation. Duration curves for an average year of the Hudson and Genesee Rivers, typical of the two areas, are compared on Fig. 5. The physical features of the Adirondack Plateau are also more favorable to water-power development, as there is an abrupt drop from the high plateau, generally in a succession of falls or steep rapids, whereas the descent from the Western Plateau, particularly on its southern side, is much more gradual.

The majority of the interior water powers, both developed and potential, are, therefore, on streams rising in the Adirondack Plateau.

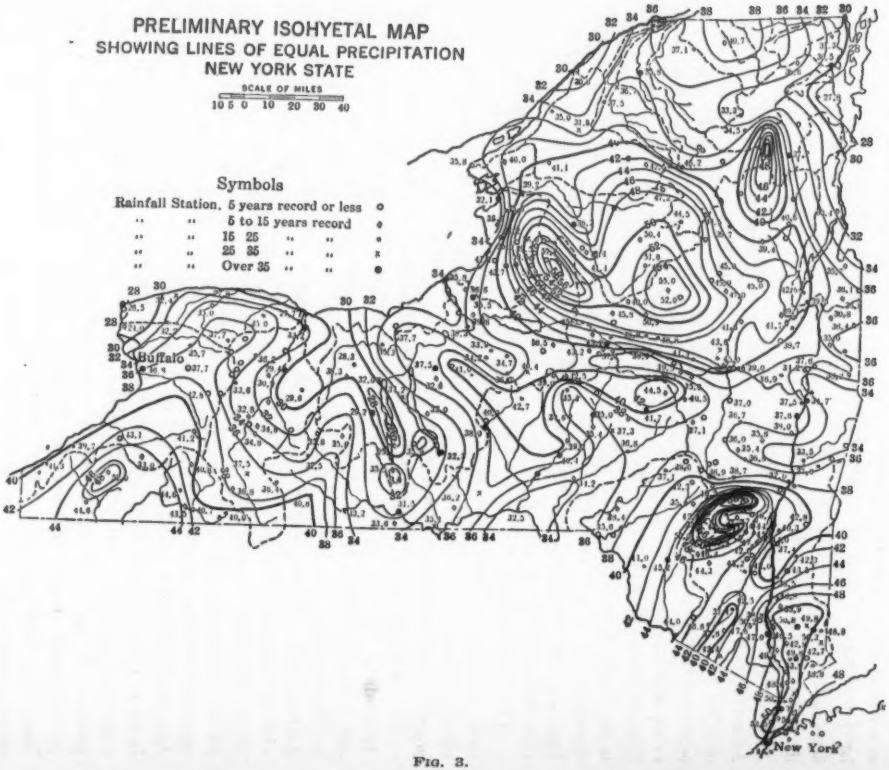

PRELIMINARY ISOHYETAL MAP
SHOWING LINES OF EQUAL PRECIPITATION
NEW YORK STATE
SCALE OF MILES
10 5 0 10 20 30 40
Symbols
Rainfall Station, 5 years record or less
" " 5 to 15 years record
" " 15 25 " "
" " 25 35 " "
" " Over 35 " "
Buffalo
New York

FIG. 3.

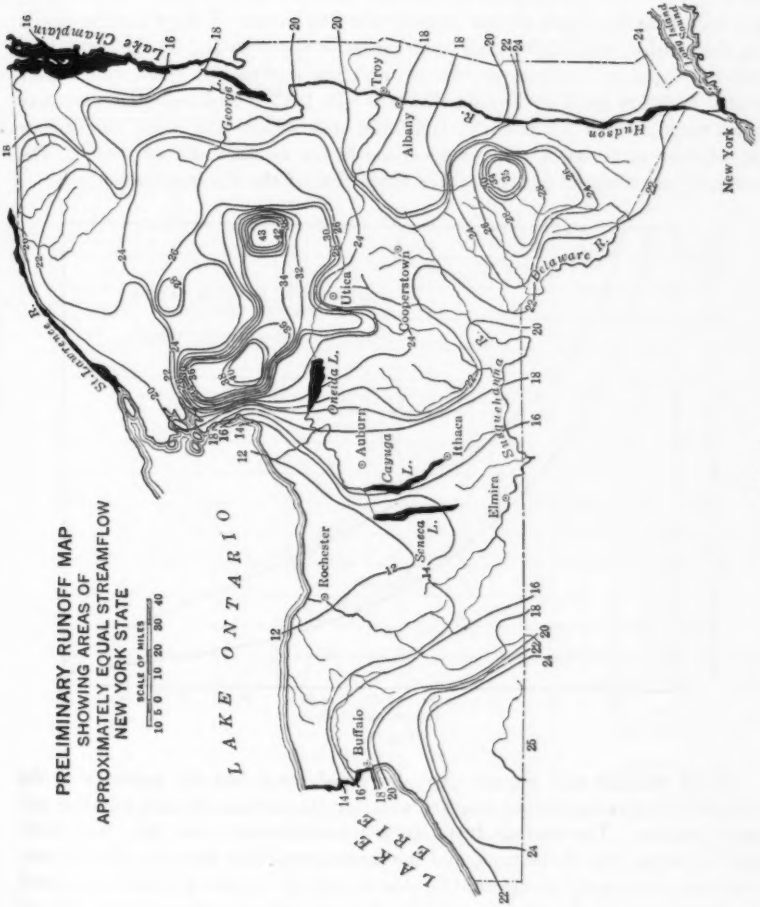

PRELIMINARY RUNOFF MAP
SHOWING AREAS OF
APPROXIMATELY EQUAL STREAMFLOW
NEW YORK STATE
SCALE OF MILES
10 5 0 10 20 30 40
LAKE ONTARIO
LAKE ERIE
Lake Champlain
St. Lawrence R.
Oneida L.
Cayuga L.
Seneca L.
Rochester
Buffalo
Auburn
Ithaca
Utica
Cooperstown
Troy
Albany
Hudson R.
Delaware R.
Susquehanna R.
Elmira
Binghamton
New York

FIG. 4.

FIG. 3.

Historical

The development of small mill powers in New York State dates back to the Eighteenth Century. The first large development in the State was started at Cohoes, on the Mohawk River, in 1833. This is the finest power site on the interior streams and has a total available fall of about 95 ft. and a drainage area of 3 600 sq. miles, giving an average annual run-off of about 6 000 cu. ft. per sec. The development was accomplished by means of three parallel canals on the hillside, with differences in elevation of about 20 ft. About 12 000 h. p. was developed by turbines or wheels operating mechanical drive, and a great textile industry grew up around this site. In 1917, a hydro-electric plant was substituted, which utilizes the full head and which will have an ultimate installation of 50 000 h. p., corresponding to the average annual run-off. The average power output is about three times that of the old installation.

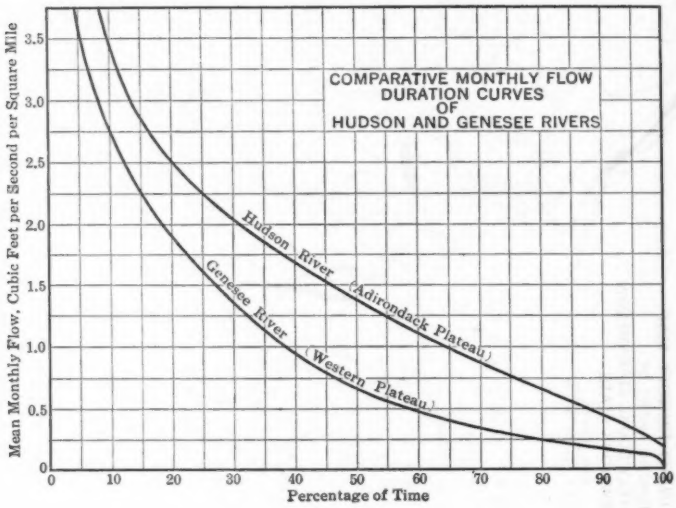


Fig. 5.

Other smaller mill powers were developed later, but the majority of the installations during the last century were for the purpose of pulp grinding and paper-making. The average installation was sufficient to use the water available for 60 to 70% of the time, and the intermittent flow was not of much consequence as the pulp wood could be stored, and the grinding (which consumed the larger part of the power) could be done at periods of high water. By the end of the Nineteenth Century, the water-power installations on the interior streams were in excess of 400 000 h. p.

The first large hydro-electric installation was at Spier Falls on the Hudson and was completed in 1903. The total head is 79 ft., the drainage area, 2 800 sq. miles, and the average annual run-off, about 5 200 cu. ft. per sec. The installation of eight horizontal wheels and generators is typical of the period,

but rather in excess of the average installation. It is capable of using the average annual run-off of 5 200 sec-ft. which is available for only 30% of the time in the average year (Fig. 6). It is believed that the size of the initial installation (considered excessive in those days) was dictated by financial reasons rather than sound economic policy. Considering the large pondage available at this plant (50 000 000 cu. ft. with a 4-ft. draft), this installation would be considered normal to-day.

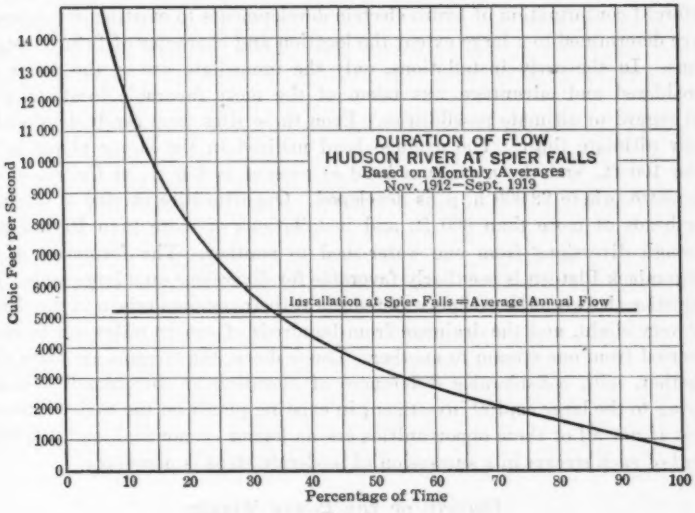


FIG. 6.

About 1903, the Indian Lake Reservoir, at the head-waters of the Hudson, having a capacity of 5 000 000 000 cu. ft. (115 000 acre-ft.), was constructed by an association of the power owners with the co-operation of the State. This reservoir and one on Salmon River, with a capacity of about 3 000 000 000 cu. ft., are the only storage reservoirs of any size, that have been constructed in New York State for power purposes. Small reservoirs have been built from time to time by private companies, and existing lakes and ponds have been drawn on during the dry months. The State also, from time to time, has constructed a number of large and small reservoirs to compensate for the diversion of waters for canal purposes. Power owners have benefited from such reservoirs, but private owners have done little or nothing to improve their powers by storage. One reason for this is the fact that, until recently, mechanical installations for paper-making were in the majority on interior streams, which installations have no great need for firm power or for storage, except in sufficient quantity to compensate for extreme low flows that might interfere with operations other than pulp grinding.

Since the construction of the Spier Falls Plant, nearly all new installations on interior streams have been hydro-electric, as well as certain of the additions

to existing mechanical plants. There has also been some substitution of hydro-electrical installation for mechanical plants so that, to-day, on the interior streams, there is about 400 000 h. p. in mechanical installations and about 500 000 h. p. in hydro-electric plants.

Present Situation with Regard to Water-Power Development

The development of more than 400 000 mechanical h. p. on the interior streams of New York State, prior to the introduction of electricity, and the enforced conformation of hydro-electric developments to existing developments have determined to a large extent the location and character of future installations. In the early installations, only the immediate use of the power was considered and advantage was taken of the most favorable locations without regard to ultimate possibilities. Even these sites were rarely developed to their ultimate limit. The average head utilized in the larger plants is less than 100 ft., and the maximum head at present is 270 ft. at Colton, on the Raquette, where 22 000 h. p. is developed. Opportunities existed at one time for heads of more than 600 ft. and installations of more than 100 000 h. p. through diversions from one water-shed to another. The formation of the Adirondack Plateau is peculiarly favorable for diversions on a large scale. The definitions between the various water-sheds at their common origin in the plateau are very slight, and the drainage from hundreds of square miles can be easily diverted from one stream to another. Lower down, the streams are often close together, with considerable differences of elevation at corresponding points. Owing to the large capital investment in existing plants on the various streams, most if not all of these opportunities are no longer economical, and a development of each stream in a succession of moderate steps is necessary.

Growth of the Power Market

The growth of the power market in New York State has been typical of that in the older industrial communities. First, there was a period in which small detached units were developed, followed by one in which gradual combination and expansion into local systems took place. The latter was followed by a highly competitive period resulting in absorption of some of the smaller plants and in consolidation and determination of territory. The combination of local companies into larger systems, which has been characteristic of some other regions, has not yet made much progress in this State. As a result, there is only one large distributing system in the State and only one transmission line of more than 66 000 volts, although a 110 000-volt line is now in process of construction by the Niagara, Lockport and Ontario Company, the distributing system previously mentioned. Even in Greater New York, three separate commercial companies and five different steam or rapid transit railways are producing power independently. In the last few years, a number of local interconnections have been made for the interchange of power, but, in general, there are a large number of strong local systems with the power generated either by steam or water within the district served. Therefore, even in the same territory, numerous differences in frequencies and in generating, transmitting, and distributing voltages have grown up between the

different systems and, even within systems, all stages of progress in the electrical art are sometimes illustrated. One small system actually uses all three frequencies as well as direct current.

Present Power Market

The power market of the State is varied in different localities, but may be divided into five groups:

First.—The region immediately surrounding Niagara Falls where the load is heavy industrial and electro-chemical, supplied entirely by water power, with an annual load factor of between 80 and 90 per cent.

Second.—Buffalo, and that part of the State west of Syracuse served by water power from Niagara, with a certain amount of steam auxiliary power for meeting peaks. The load is industrial and commercial, and the annual load factor is between 60 and 65 per cent.

Third.—The northern slopes of the Adirondacks, served entirely by water power, either from the interior streams or from the St. Lawrence. The load is largely industrial and the annual load factor is about 60 per cent.

Fourth.—The New York Metropolitan District, where the load is commercial, light industrial, and railway. All the electric power in this district is steam generated, and the combined load factor is about 40 per cent.

Fifth.—The remainder of the State, principally the valleys of the Hudson and Mohawk, served by intermittent water powers on the interior streams and a considerable amount of steam auxiliary. The load is mixed industrial and commercial, and the annual load factor is about 50 per cent.

To summarize: Outside New York City, the main source of electrical energy is water power, and the load is chiefly industrial. Only in New York City does the lighting part of the commercial load play an important part.

The average annual load factor for the entire State is 42%, and complete interconnection would result in a load factor of about 50%, indicating a diversity of about 16%, as follows:

A	= Sum of non-coincident peak.....	1 708 500
B	= Coincident peak	1 464 900
C	= Diversity	16.6%

$$C = \frac{(A - B)}{B} \times 100$$

The characteristics of the ideal load that would be obtained by complete interconnection are shown on the duration curve, Fig. 7 and the peak day-load curve, Fig. 8. The theoretical saving in capacity that would result from interconnection would be about 250 000 kw., exclusive of the saving in reserve capacity which would inevitably follow. Other considerations, nevertheless, compel the conclusion that the economies in capital expenditure and operating expenses due to interconnection, in themselves, would not warrant the necessary expenditures for transmission. If, however, the further development and distribution of water powers should warrant the construction of trunk transmissions, great additional benefits would follow through the interconnections thus caused.

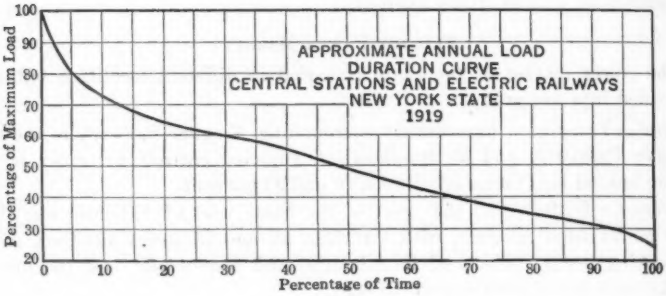

APPROXIMATE ANNUAL LOAD
DURATION CURVE
CENTRAL STATIONS AND ELECTRIC RAILWAYS
NEW YORK STATE
1919
Percentage of Maximum Load
Percentage of Time
100
90
80
70
60
50
40
30
20
0
10
20
30
40
50
60
70
80
90
100

FIG. 7.

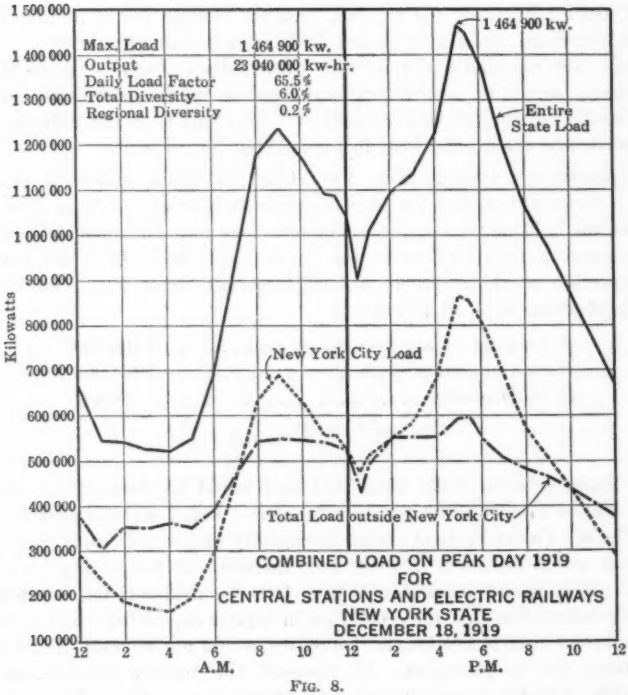

Max. Load 1 464 900 kw.
Output 23 040 000 kw-hr.
Daily Load Factor 65.5 %
Total Diversity 6.0 %
Regional Diversity 0.2 %
1 464 900 kw.
Entire State Load
New York City Load
Total Load outside New York City
COMBINED LOAD ON PEAK DAY 1919
FOR
CENTRAL STATIONS AND ELECTRIC RAILWAYS
NEW YORK STATE
DECEMBER 18, 1919
Kilowatts
1 500 000
1 400 000
1 300 000
1 200 000
1 100 000
1 000 000
900 000
800 000
700 000
600 000
500 000
400 000
300 000
200 000
100 000
12
2
4
6
8
10
12
2
4
6
8
10
12
A.M.
P.M.

FIG. 8.

Frequency

The chief obstacle to complete interconnection and interchange of power is the difference in frequency. In 1919, the total generating capacity in central stations and electric railways throughout the State was divided, as shown in Table 1.

TABLE 1.

Frequency.	Capacity, in kilowatts.	Percentage of whole.
25-cycle	1 464 600	67.2
40-cycle	116 400	5.3
60-cycle	516 000	23.6
Direct-current and miscellaneous	86 100	3.9
Total	2 183 100	100

A large part of the current generated as 25-cycle is utilized as direct current, whereas the direct current generated is utilized almost entirely in manufacturing processes. All the 40-cycle is concentrated in the Albany-Schenectady District. The 25-cycle is almost entirely concentrated in New York City and in the territory west of Syracuse served by the Niagara water powers. There is a large and growing 60-cycle capacity in the New York City Metropolitan District and throughout the remainder of the State, 60-cycle is in a rapidly growing ascendency.

The tendency is toward the gradual elimination of the 40-cycle frequency in favor of the 60-cycle and the restriction of the 25-cycle to territory now served. It is doubtful whether the 60-cycle frequency will ever gain any great foothold in the Niagara District, but the next ten years should witness a great reduction in the 40-cycle, the increase of the 60-cycle to at least a parity with the 25-cycle in New York City, and the virtual elimination of the 25-cycle frequency except in New York City and the Niagara District.

Present Tendencies

The present tendencies in water-power development in the State of New York are:

First.—The substitution of hydro-electric for mechanical power in existing plants.

Second.—Plans for the provision of greatly increased amounts of storage.

Third.—The selective development and operation of water-power plants of varying characteristics.

Fourth.—Plans for the unified development of individual streams so as to utilize maximum possibilities.

Fifth.—Increased interconnection to permit of profitable interchange of power between plants of different characteristics.

Sixth.—Greater over-development for the purpose of meeting the market load factor and a decrease in the number of units.

Change from Mechanical to Hydro-Electric Power

The change from mechanical to hydro-electric power has been proceeding gradually and is almost completed in all industrial plants except those engaged in wood-working and paper-making. The principal use of power in paper-making (which utilizes more than three-quarters of the present mechanical power) is for pulp grinding, and there is practically no advantage in electrical installations, except in the possible disposal of surplus power. A number of factors are now tending against a continuance of the paper industry in New York State, among which may be mentioned the decreased supply of available timber and the increased cost of getting it to the mill, the increased value of hydro-electric power, due to the increased cost of producing power by steam, the growth of the electric power market in the immediate vicinity, the development of cheaper water power in Canada adjacent to plentiful supplies of pulp wood, and the operation of the present tariff. It may be safely predicted that the rate of change from mechanical to hydro-electric power will be greatly accelerated during the next decade. This power will seek an outlet in the growing public utility market and will tend to diminish the new hydro-electric construction.

Storage

Storage developments for power purposes in New York State, to date, have been insignificant. With about 900 000 h. p. installed on the interior streams, the total storage is only 20 400 000 000 cu. ft. (470 000 acre-ft.), or less than 1½% of the annual run-off. Of this, 14 300 000 000 cu. ft. has been provided by the State as compensation for canal diversion, leaving only 6 100 000 000 cu. ft. which has been provided by power companies for their own use. The Adirondack Plateau is a lake region with numerous large bodies of water. Unfortunately, the use of most of these lakes for regulation is greatly limited or prevented by the settlement along their shores of numerous summer colonies with expensive improvements. However, almost countless natural basins exist in thinly settled or wild localities, that can be economically improved. Within the Adirondack District, it has been estimated that there are 59 practicable reservoir sites with a total capacity of about 230 000 000 000 cu. ft. (5 300 000 acre-ft.), which is sufficient to store 32% of the average annual run-off in the District. It is physically possible, therefore, to control all the Adirondack streams to such an extent as to make them independent of any steam auxiliary, if this should prove desirable and economical. The limits of this paper do not permit discussion of the economic quantity of storage, but, at least, a certain proportion of these reservoirs are surprisingly cheap in view of the benefits that would be reaped.

There are many reasons why more storage has not been developed, but the chief reasons are as follows:

First (and Foremost).—The ownership, by the State, of 2 000 000 acres of land scattered through the Adirondack District. This constitutes the Forest Preserve and, by a provision of the Constitution, it can neither be sold nor used. There are only a few possible sites that do not contain at least one parcel of State land. It was not until 1916 that an amendment to the Con-

stitution, permitting the use of not more than 3% of these lands for storage purposes, was followed by the necessary legislation authorizing the organization of public corporations known as River Regulating Districts to take over such lands as are needed for the reservoirs. The intervention of the World War and the necessity of perfecting legislation have delayed action on any large scale, but one such district is now in successful operation and legal restrictions are now removed.

Second.—The co-operation between numerous independent owners of developed plants on each stream in the administration and expense of the storage reservoirs has been difficult to obtain in many instances.

Third.—A considerable proportion of the independent owners are paper-makers using mechanical drive. Most of their power is consumed in pulp grinding which need not be carried on continuously as the pulp wood is floated down on the spring floods and, in any case, must be stored. Grinding, therefore, can be done when there is sufficient water, and the only need of regulation is to prevent extreme periods of low water which may interfere with their other operations. They require, therefore, only a low degree of regulation as opposed to the high degree of regulation which is economical for the hydro-electric plant. However, as the average paper-making plant is installed for the flow available for 60 to 70% of the time and, for hydro-electric plants, the present economical installation is for the average flow or that available for 30% of the time, it is possible by adding an equal installation of hydro-electric apparatus at the paper plant to enable the combined installation to reap the full benefit of the high degree of regulation. The average small paper-maker is, however, loath to enter the hydro-electric field which he does not understand and in which he controls no market. The influence of the paper industry, therefore, has heretofore been against large amounts of storage and the gradual decline of this industry will accelerate the storage program.

Public and private investigations of the storage possibilities of the State have been made sufficient to establish the physical and financial features of the storage situations. As the way is now open from a legal point of view, and both public opinion and ideas within the utility industry have been crystallized, it would appear that a period of extensive storage development is now possible.

Selective Development and Operation of Water Powers

In New York State, there are two main classes of water powers to consider, that is, the continuous powers in the boundary streams and the intermittent powers on the interior streams.

The continuous powers were developed at first for continuous loads, such as those of the metallurgical and electro-chemical processes with which annual load factors of 80 and 90% are common. When the continuous powers were introduced to the general market with a prevailing load factor of 50 to 60%, it was found profitable to use these as far as possible on the base load and to obtain peak capacity from other sources. Thus, at Buffalo, only 20 miles from Niagara, is located the 100 000-kw., steam plant of the Buffalo General Electric Company, which operates in conjunction with continuous water power purchased from Niagara. The technical reasons for the construction and opera-

tion of this steam plant have never been clear in the public mind, and the charge is frequently made in the press that Buffalo has been deprived of her fair share of Niagara power. Attempts at denial or explanation are met by the statement, which is usually conclusive to the lay mind, that it has been necessary to erect a large steam plant in Buffalo. Nevertheless, if the continuous powers are to be utilized in the general market to the best advantage and with the greatest economy, it is certain that peak power must be supplied either from steam or from the intermittent interior water powers.

The interior water powers, owing to lack of storage, are at present dependent on large steam auxiliaries. This steam auxiliary makes possible a certain variety of operation, either carrying all the load by water in periods of high flow, or carrying base or peak loads by water or steam. When adequate storage regulation has been provided, it will be theoretically possible to operate without steam, although it is the speaker's opinion that a certain amount of steam power will always be utilized for regulation, flexibility, and occasional peaks. With complete storage regulation, it will be further possible for the interior water powers to be used as auxiliaries to the large continuous powers or to some large steam-power market, such as New York City. For working within a system by themselves, it will be possible to adapt the design of the different plants to the physical conditions. For example, a plant with good pondage, high head, and short pipe lines or penstocks may be greatly over-installed and used on the peak load, whereas plants of opposite characteristics may be operated continuously on the base load. It may be that certain streams can be devoted entirely to the production of peak power and others to that of base power.

An example of the harmonious development and operation of continuous water power, intermittent water powers with ample storage, and steam, is furnished by the system of the Niagara, Lockport and Ontario Company. The conditions in 1916 were, as follows:

Maximum 15-min. integrated peak.....	82 400 kw.
Total output for the year.....	427 045 000 kw-hr.
Average load factor for the year.....	about 57%.
Capacity:	
Continuous hydro-electric power purchased from Niagara.....	50 000 kw.
Installed capacity of hydro-electric power on Salmon River.....	32 000 "
Installed capacity of hydro-electric power at Minetto (Oswego), N. Y.....	9 600 "
Installed capacity of steam electric power at Lyons, N. Y.....	29 500 "
Total capacity.....	121 100 kw.

All these plants are tied together by transmission lines nearly 200 miles long and are operated as a unit. If considered as independent plants designed to operate continuously on a normal load factor, the machine installations at

Salmon River and, to a smaller degree, at Minetto, are excessive. A normal installation for independent operation at Salmon River, considering the water available, would be not greater than 11 000 or 12 000 kw., as compared with the 32 000 kw. actually installed, and at Minetto not greater than 6 000 kw., as compared with the 9 600 kw. installed. This over-installation is the basis of the methods of operation.

Methods of Operation

The continuous water power, purchased at a flat rate, is operated on the base load at a high load factor.

The Minetto plant is operated generally during the day on the base load, with a high load factor while operating.

The peak load is taken by the Salmon River plant, aided by steam in seasons of low water, both operating on a low load factor.

Results

During 1916, the continuous water power had a load factor for the year of nearly 90%, and a consequent use of about 90% of the available water. The steam and water power operating on the peak had a load factor for the year of about 22 per cent. Owing to the great quantity of storage at the Salmon River plant (2 900 000 000 cu. ft.) and to over-installation permitting intermittent operation, almost 90% of the available water in this stream was used.

During 1919, only 31 000 000 kw-hr. of a total of 435 000 000 kw-hr., or 7% of the total power was furnished by steam. The result is that, with a system load factor of less than 60%, the load factor on water power while operating is nearly 90% and, consequently, nearly 90% of the available water is used. The extra expense chargeable to this method of operation is the cost of the additional machine installation at Salmon River as the transmission lines are necessary in any case as distributors. The fixed charges on this additional installation are only a fraction of the value of the increased output.

The future of water-power development in New York State lies in the provision of adequate storage for the interior streams and in the selective development and operation of the plants on these interior streams in harmony with the great continuous powers of the St. Lawrence and Niagara. Although the cost of development on the interior streams is uniformly greater, the ability to furnish peak power makes their output more valuable.

Unified Development of Individual Streams

These possibilities can be realized to their fullest extent only by stream regulation and the harmonious development and operation of a number of plants on a stream, or group of streams, in accordance with a well ordered plan, and by providing the necessary transmission ties, so that operation can be based on a community of interest and that development will no longer be governed by local considerations or the immediate market.

Considering the age of the water-power industry in the State, and the fact that it was well established on a number of the interior rivers prior to

the introduction of hydro-electric power, it is not surprising to find the greatest variety of ideas in the design of the different plants on a single stream. The mechanical and hydro-electric plants are interspersed, and the desirable methods of operation of the two types are dissimilar. Consequently, there is considerable interference at times. The ponding of water at night by the hydro-electric plants, if carried too far, tends to hamper continuity of operation in the mechanical plants, and as the installations of the latter are generally much smaller in proportion, may even cause considerable loss of power. Until recently, water-power legislation in the State was backward and the Courts usually relied on an interpretation of the common law by which every owner of riparian rights was entitled to the natural uninterrupted flow. Any harmonious scheme of development and operation, therefore, was dependent on unanimous private agreement which has never been accomplished. Interference between adjacent plants has generally been the subject of private agreement and mutual compromise, but on no stream of any size has a unified plan been devised and adopted for harmonizing the size of the installations and methods of operation of the various plants.

The organization of the River Regulating District based on a mutual understanding between all the power owners on a stream, has changed this situation, and the gradual conversion of the mechanical plants to hydro-electric power is removing the chief source of disagreement. The possibility of supplementing pondage by the release of water from the reservoirs during certain hours of the day, thus providing additional peak capacity, has drawn further attention to the necessity for unified operation. Plans have been prepared for the re-installation of a series of plants on one stream with a view to the operation of all of them for the maximum total output and the best operating conditions in the combination without regard to the individual plants. It may be stated, therefore, that there is a distinct tendency toward more harmonious development and operation.

Interconnection

For a long time, the method of growth of the power industry in strong local systems already described and the differences in frequency and voltage tended to minimize interconnection. Even between systems of the same nominal frequency there were often differences in actual frequency. The World War showed the importance of interconnection in permitting transfer of blocks of power and tended to overcome some of the differences. Interconnections and interchanges brought about by the war were often found to be profitable, owing to diversity of load or to the operating conditions surrounding certain water-power plants. For example, water-power plants with large pondage found it profitable to shut down at night during periods of low water and take surplus power from plants with small pondage, sometimes returning peak power to the latter by day. Local interconnection for such exchange has made great progress recently. Several trunk transmission lines of moderate length and voltage are now in course of construction or rehabilitation. It would seem that any general scheme of interconnection such as that

proposed by the Super-Power Survey is dependent on the development of water power on a large scale.

Over-Development and Decrease in Number of Units

On an unregulated stream, the present normal installation in hydro-electric plants is about sufficient to use the average flow. On the Adirondack streams this will permit, during the average year, continuous operation for only 30% of the time; but continuous operation during a 10-hour day is assured for 65% of the time for the average year. This requires the pondage to be sufficient to conserve the night flow. Where no pondage is available, the installation is generally less.

As the minimum flow is increased by regulation, pondage requirements become somewhat greater, unless the reservoirs can be operated on an hourly schedule. On a fully regulated stream, the installation may be twice the minimum regulated flow available for 95% of the time, or even higher where ample pondage is available. This will permit of operation on an annual market-load factor of less than 50 per cent. The general tendency of the past few years has been toward greater installation, and this undoubtedly will be accentuated by regulation.

Number of Units

A time-honored rule among steam power plant designers is that an independent power plant should contain from five to eight units. In the early hydro-electric plants, of which Spier Falls is a type, eight units were used. Since that time, there has been an increase in the size of units and a consequent reduction in number. The principal limiting conditions in determining the number are the maximum economical size, the flexibility necessary for meeting load conditions, and the necessity for break-down reserve. With the growth of the market, the increase in the number of water-power plants in a system, and growing interconnection, the requirements for flexibility and break-down reserve have decreased. Sufficient flexibility for economic operation can usually be obtained with three units. The maximum number of units in plants of modern design is five and the minimum two. In general, it may be stated that present practice calls for three to five units, with a tendency toward the smaller number wherever the economic limit of size of unit will permit, and this will undoubtedly be accentuated by the tendency toward unified development and harmonious operation of all the plants on each stream.

Conclusion

In no part of the United States has conservation of natural resources through the development of water power been given more thorough and competent public and private study than in the State of New York. Nevertheless, during the past few years, development has lagged, and the additions which have been made, have been the result of individual initiative. Legal restrictions which have been largely responsible for the lack of accomplishment, have now been removed, and the speaker believes that the State of New York

is on the eve of a comprehensive development of the natural resources on the interior streams. In order to obtain the greatest benefit, such a development must be based on a community of interest and thoroughly co-ordinated, in order that the maximum economic possibilities of each stream may be eventually realized. This means that there must be a general plan and that each individual step whether dictated by public or private initiative must fit into the general plan. Such a result can be realized only by the development of the spirit of co-operation.

THE OPERATION OF THE FEDERAL WATER POWER ACT

By O. C. Merrill,* M. Am. Soc. C. E.

Probably more water-power projects are now under construction than ever before in the United States, and the Pacific Coast, as usual, is leading in this work. One of the most important elements in this renewed activity is the Federal Water Power Act which was passed by Congress in 1920. Perhaps the time has been too short to permit of passing conclusive judgment either on the law or on its administration, but the fact that the applications filed with the Commission aggregate more than 20 000 000 h. p., and that the greater part of the projects now under way are being built under the provisions of the Act, should afford at least a tentative basis for appraising its value.

The early years of the operation of any statute are particularly important, because they are the period of its interpretation and of the establishment of precedents in its administration. For this reason, and also because this particular statute has so close a relation to the work of the engineer, it is desirable that all engineers should be familiar with its essential features and fundamental bases, with the manner in which it is operating, and with the results that are being accomplished.

Many members of the Society are familiar with the water-power situation as it existed before the present law was passed, and with the difficulties and uncertainties that surrounded any attempt to make use of the resources that came within the jurisdiction of the Government. The United States is the owner of the public lands, and these lands may be used only in such manner and for such purposes as Congress may direct. Through its power to regulate commerce, Congress has jurisdiction over all the navigable waters of the United States and may determine what structures may be erected in or over them and under what conditions. The manner in which international waters may be diverted and used is the subject of treaty between the nations concerned, and the sole power, in this country, of making and enforcing treaties is the Federal Government. These are the three fundamental bases on which rests the authority of Congress over water powers. Under this authority, it has control over the disposition of about 85% of the potential water powers of the United States. It is of great importance, therefore, that the legislation which Congress enacts shall be wisely conceived and intelligently administered.

It was the early policy of Congress to leave the regulation of navigable rivers, and, in a large measure, their improvement, to the several States and to acquiesce in the construction in such streams of whatever structures the State laws might authorize. A similar policy was first pursued with respect to the use of public lands for power purposes. This early policy was gradually modified through legislation which sought to preserve the public interest in the power resources under public control, but which failed to appreciate the conditions which were necessary if these resources were to be developed for public use. In fact, the Act of 1901 which applied to the public lands, and

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the Act of 1906 as amended in 1910, which applied to navigable rivers, and which together constituted the legislation in effect when the Federal Water Power Act was passed, were adopted at too early a stage in the history of hydro-electric development to be suited to present-day conditions.

The Act of 1901 expressly limited the rights obtainable on the public lands to mere revocable licenses; and even these rights were jeopardized by the possibility that the lands to which they applied might be patented to others. Authority was granted in the form of permits issued under general regulations prescribed by the Executive Departments. The Act itself contained no expression of policy, and the regulations issued reflected the views of administrative officers and were subject to change at discretion. The Act of 1910 expressed the general conditions under which rights on navigable rivers could be secured. The grant itself required in each instance a special Act of Congress and was limited to fifty years. Plans were subject to approval by Federal authority, and the construction of locks or other navigation facilities might be required either at the time of original construction or at some indefinite time in the future. The failure of both Acts to make any provision for disposition of the properties or for extensions of the rights on termination, and the reserved right to revoke any permit or to alter or repeal any special act at any time without recourse in the grantee or liability in the United States, made financing almost impossible.

As a result of this unsatisfactory legislation only about 800 000 h. p., largely in extensions of existing projects, was constructed on the public lands and reservations under the Act of 1901, and about 500 000 h. p. on navigable streams under the provisions of special and general legislation. Of the latter amount, only 63 300 h. p. had been developed in 10 years under and following the Act of 1910. While six-sevenths of the nation's potential water-power resources were under Federal jurisdiction, scarcely more than one-seventh of the existing developments had made use of sites under Government control. It became apparent, therefore, that any considerable utilization of these sites would require new legislation, and that such legislation must give consideration to the investment in the properties as well as to the rights of the public. The Federal Water Power Act is the result of the efforts to strike a balance between these two interests.

In place of the uncertain tenure and unknown requirements of previous legislation, an applicant for a power project under the Federal Water Power Act, may secure a license for a period up to fifty years. The license is a contract between the Government and the licensee. It expresses all the conditions which the licensee must fulfill and, except for breach of conditions, cannot be altered during its term, either by the Executive or by Congress, without the consent of the licensee. If a licensee fails to commence construction, the license may be canceled by administrative action; but after construction has been started it can be canceled only by judicial action, and only then if no other appropriate legal remedy is available. When the license period expires, the Government may take over the properties for its own use, may permit them to be taken by another, or may issue a new license to the original licensee.

If the properties are taken over before the term of the license expires, "just compensation" must be paid, as determined by the Courts in condemnation proceedings. If taken over at the termination of the license, the price is the "net investment," an amount which represents the original cost less so much of the then existing reserves as have been accumulated out of earnings, in addition to a fair return. If the properties are not taken over by the Government or by another, the licensee is entitled to a new license on such terms as will be reasonable in view of conditions then existing. In so far, therefore, as action of the Federal Government is concerned, full recognition is given to every dollar of honest investment, but to no more.

A licensee under the Act, however, has obligations as well as rights, but these obligations have for their purpose the creation of conditions under which electric energy can be produced and sold at the least amount consistent with the protection of the investment and a reasonable return. The Act requires that works be designed with due regard to safety and efficiency and to the fullest practicable utilization of the water power available, that they be maintained in full operating efficiency, and that all necessary renewals and replacements be made. It requires that reserves be established out of earnings, in order that assets or credits may be available from which replacements can be made and by which the investment may be kept intact. The Act also requires the maintenance of such a system of accounts as may be prescribed by the Commission, in order that information may be had with respect to investment and earnings, and that when the license expires records may be available adequate for determining the terms of settlement, should the properties be taken away, or the terms of extension, should the license be renewed.

The Act provides for permanent public ownership and control of power sites on public lands and of power privileges in navigable and international streams, and for their development by private capital under full public supervision and regulation. It prohibits capitalization for purposes of rate-making or of public purchase of rights, franchises, lands, or other properties, in excess of their legitimate cost. It requires that compensation be made for rights granted and for the use of lands or other properties of the United States; and it provides that a part of any earnings in excess of a specified reasonable rate of return shall be used in retirement of the investment in the properties.

That these provisions of the Act are adequate to protect every legitimate public interest is evident from their mere recital. That the Act also affords conditions under which capital may be secured for development purposes is evidenced by the results accomplished in the 2¼ years in which the Act has been in effect. In this time, 348 applications for permits or licenses, involving an aggregate estimated installation in excess of 20 000 000 h. p., have been filed with the Commission. This amount is more than twice the existing water-power installation of the United States. It exceeds the combined potential water-power resources of Norway, Sweden, Finland, and the Arctic and Baltic drainages of Russia—the chief water-power region of Europe. It is nearly twice the combined resources of France and Italy. It is more than six times the aggregate of all applications for power sites under Federal control in the preceding 20 years. To October 1, 1922, the Commission had author-

ized 64 preliminary permits and 64 licenses, of which 25 were for transmission lines. The 60 permits now outstanding involve an estimated installation of 2 540 000 h. p., and the 39 licenses for power projects, 2 040 000 h. p., or a total of 4 580 000 h. p. Of the projects covered by the 39 licenses, 15, involving an estimated installation when completed of 1 880 000 h. p. and investments of not less than $250 000 000, are either completed or under construction. This is one and one-half times as much as was constructed under Federal authorization in the 20 years preceding the passage of the Federal Water Power Act. As a practical result of the present law, probably more water power is now under construction than at any previous period.

The problems in the administration of the Act are many and varied. Duties have been placed on the Commission not hitherto exercised by the Executive Departments. Policies and practices must be developed to meet the new situation created by the passage of the Act. The Act itself has required interpretation, and decisions have been rendered respecting the relations of the Commission to other agencies, State and Federal, and the bearing of State and Federal laws and international treaties on the work of the Commission. No attempt will be made in this paper to discuss these matters, nor the various duties of the Commission, in any detail. The speaker does wish, however, to consider two questions relating to the Commission's jurisdiction, which are of major importance, for on its policies in this respect will depend in no small degree the successful administration of the Act. Whether any particular project must secure a license, and what is the line of demarcation between the regulatory—or supervisory—powers of the Commission and those of the several States, are the two chief jurisdictional questions. Neither is completely answered by the Act itself, and neither could be, without depriving the Commission of that discretionary power necessary for dealing intelligently with the particular facts of individual cases.

When power projects involve lands or other property of the United States, the jurisdiction of the Commission admits of no doubt. Ownership is a question of fact, and when the fact is established, the authority of the Commission is clear and exclusive. Outside the public lands and reservations, the jurisdiction of the Commission involves two classes of streams: First, those streams which are defined in the act as "navigable waters," over which the Commission has direct jurisdiction, and in which development cannot lawfully be made without its prior approval; and, second, those non-navigable tributaries of navigable waters in which power development, by altering the natural flow, would affect the navigable capacity of the navigable waters. The second class comes under the jurisdiction of the Commission only when declarations of intention to construct dams within such streams are filed with the Commission.

The construction of power projects on streams of the first class, that is "navigable waters," without first securing a license under the Federal Water Power Act, would be subject to the penalties prescribed by that Act and by the Act of March 3, 1899, which prohibits without prior approval "the creation of any obstruction * * * to the navigable capacity of any of the waters of the United States."

The Federal Water Power Act defines navigable waters as:

"* * * those parts of streams or other bodies of water over which Congress has jurisdiction under its authority to regulate commerce with foreign nations and among the several States, and which either in their natural or improved condition notwithstanding interruptions between the navigable parts of such streams or waters by falls, shallows, or rapids compelling land carriage, are used or suitable for use for the transportation of persons or property in interstate or foreign commerce, including therein all such interrupting falls, shallows, or rapids; together with such other parts of streams as shall have been authorized by Congress for improvement by the United States or shall have been recommended to Congress for such improvement after investigation under its authority."

Whether a stream has been recommended for improvement, or improvement has been authorized or actually made, is a simple question of fact. Whether a stream is used in the transportation of persons or property in interstate or foreign commerce is also a question of fact which may be determined on investigation; but whether, if so, the amount or character of such commerce is sufficient to warrant the assertion of jurisdiction by the Commission is a matter for decision in the individual case. Similarly, whether a stream which is not now used in interstate or foreign commerce, is suitable for such use, is a matter of judgment, and decision must rest on the character of the stream and on the probability of the future development of commerce.

The Act provides a procedure whereby any one proposing to build a dam in any stream of doubtful status may make declaration of his intention and have the matter conclusively determined by the Commission. There has been criticism, particularly in the New England States, that the definition contained in the Act is not clear and that the Act should be modified or the definition interpreted by the Commission so that the limit of navigability on any stream may be determined with such exactness that no one need ever be in doubt or need ever apply to the Commission for its determination.

The question of navigability is not one of mathematical formulas, and there is no more probability that it can be removed from the domain of individual judgment than that laws in general can be drawn with such precision that the services of the Courts can be dispensed with in their interpretation. The decisions of the Commission in the individual cases presented to it will establish precedents that will gradually clear the situation. Three recent cases, on interstate streams, the Saco River, running from New Hampshire into Maine, the Connecticut, a boundary between New Hampshire and Vermont, and the Menominee, a boundary between Wisconsin and Michigan, are indicative of the policy the Commission is pursuing. The first two are actually carrying property in interstate commerce in considerable quantities in the form of logs and pulpwood, and the third is suitable for such use. All three are, therefore, technically navigable; but due to the fact that, in each instance, the laws of the State afford full protection to such commerce, and that the proposed dams would not obstruct it, the Commission made formal finding that the streams were not "navigable waters" within the definition of the Act, that the proposed dams would not affect the interests of interstate commerce, and that, therefore, no Federal license was required for their con-

struction. The Commission does not propose to extend its jurisdiction in this respect beyond the point where some substantial interest of interstate or foreign commerce is involved.

The Commission has certain purely administrative duties. It has others which are more or less regulatory in their nature. It must be assured, for example, that the projects which it licenses, are adequately maintained and efficiently operated; that a system of accounting is established; that periodical reports are made; that adequate depreciation reserves are created and maintained; and that under certain circumstances amortization reserves are set aside. In the absence of State authority, it may regulate the rates and services and the issuance of the securities of its licensees. In some respects, its authority is exclusive; in other respects, its authority is concurrent with similar authority of the several States, and the question at once arises where should the jurisdiction of the one end and of the other begin, for it is manifestly desirable that State and Federal agencies work in harmony and that each assume responsibility for those duties which, at any particular time, it is in the best position to perform.

The fundamental difference between the regulatory power which should be exercised by State agencies and that which should be exercised by the Federal Commission is in the basis on which each rests. The former is chiefly, if not exclusively, an exercise of the police power; the latter is largely the enforcement of a contract obligation. The primary responsibilities of the Federal Power Commission, therefore, relate to the general plan of the project and its relation to other projects and other uses; to the design, maintenance, and replacement of the works; and to the determination, or to the methods of determination, of the items which make up the net investment. These are matters of supervision rather than regulation, and are to be exercised occasionally rather than constantly. The primary responsibilities of the State, on the other hand, are regulatory in nature, require continuous exercise, and have reference chiefly to the relations between the licensee and the general public. They are concerned with the quality and extent of service rendered, with the rates to be charged therefor, and, as incidental thereto, with the character and amount of securities to be issued, and with the system of accounting to be established and maintained.

In recognition of this situation, it has been the intention in preparing regulations and in adopting procedure for the administration of the Act to limit Federal supervision and control to those matters for which the Commission must itself assume responsibility under the provisions of the Act; and even within this limit to leave as far as practicable initial responsibility to the States in those matters where the latter also have jurisdiction under their own statutes, reserving to the Commission the right to take initial action in any matters not covered by State statutes, and reserving also the right to add to State requirements if such requirements do not meet those specifically imposed by the Federal statute. The Commission is not desirous of extending its own jurisdiction, but believes that the best results can be secured through co-operative action with the agencies of the States.

In addition to the duties already mentioned, the Commission is required to investigate all projects applied for, in order to determine whether the structures are safe and properly designed, and whether full utilization will be made of the resources of the stream; it must investigate and pass on applications for restorations to entry of lands in power-site reserves; and it is required to make valuations of all properties licensed under the Act and constructed prior to the issuance of a license. It is manifest that the duties placed on the Commission with respect to water powers are many times greater than have ever been assumed before by any of the agencies of the Government; and yet to perform these duties, the Commission—so the Comptroller General has ruled—may have no personnel of its own, other than its Executive Secretary and Engineer Officer, but is obliged to borrow for its work such personnel as the Departments can spare and are willing to loan. Although there is no disposition to criticize the Comptroller's decision from a legal standpoint, it is not believed that it represents what Congress intended. The Commission, nevertheless, has been operating two years under these unsatisfactory conditions with a detail of 8 engineers, 2 attorneys, 2 accountants, and 18 clerks—a force utterly inadequate to perform the duties placed on it by the Act. The Commission has no field force of its own, loaned or otherwise, but must depend for examinations and reports and for the conduct of field hearings on the field officers of the Departments—men who are primarily responsible for their own departmental duties. It was given originally an appropriation of $100 000, and this amount was repeated for 1922. For 1923, it has only the unexpended balance of its first year's appropriation; and yet, from lack of personnel, it will not be able to use even this.

Under the circumstances, the Commission has been obliged to delay action on many important projects, and it has been forced to omit altogether the performance of important duties required by the Act. This is particularly true of valuations, of which cases involving approximately $100 000 000 are now awaiting action. The Commission has been unable to secure the detail of any personnel with adequate experience in valuation work and, therefore, has been obliged either to suspend issuance of licenses where valuations are involved or to provide for valuations in the future. It has taken the latter course, in order that much needed power development might proceed. Such a course, however, is almost certain to result in prolonged litigation and in expenses many times greater than would have been required had the Commission been given the means for carrying out this requirement of the law in the beginning.

By confining its activities primarily to applications for power projects, declarations of intention, and requests for restoration to entry, the Commission has been able to take final action on more than one-half the applications for permits and licenses, or a total of 197. Of the remaining 151, 20 are not sufficiently complete for action to be taken, and action on 27 others must await prior action of other agencies. The Commission has also rendered decisions on 36 declarations of intention, passed on restorations to entry in 200 cases, and made withdrawals of 1 250 000 acres of public lands in connection with applications for power projects. Although the two years' record is reasonably satisfactory, in view of the serious handicap under which it

is working, the Commission, nevertheless, should promptly be placed in a position where it would be possible for it fully to carry out the purposes plainly expressed in the law. The present situation necessarily results in an administration of the law, which is insufficiently co-ordinated and only partly effective. The chief purpose of the creation of the Commission was to secure a common policy and a single executive agency in water-power administration. This purpose has not been accomplished. Other agencies are still continuing their independent activities, and these activities are not controlled by a common plan and are not subject to a common direction. This defect will not be cured, and the Commission will not be able to perform in full the duties placed on it, until the Act is amended so that it may employ its own personnel. It is important that this action be taken at an early date. Having waited ten long years for a legislative pronouncement of a Government policy respecting the utilization of the huge water power resources of the United States, it should not be necessary to wait another long period of years before means are provided for the execution of that policy.

There is one other matter to which attention should be directed. There are movements on foot in several quarters to secure for certain sites or streams special legislation, which, if approved, would constitute a partial repeal of the Federal Water Power Act, and eventually would result in the progressive disintegration of the present National water power policy. If these proposals that rights or authorities be granted independently of the present law, are examined, it will be found that some or all of the essential features of the Act, particularly those that protect the public interest, have been omitted, even when provisions in direct conflict have not been substituted. Furthermore, the granting of special privileges to favored interests would clearly discriminate against those who in the faith that Congress had at last fixed its policy, are investing hundreds of millions of dollars under the obligations of the Act. The results already accomplished afford convincing evidence that grants of special privileges are not necessary in order to secure the development of all the electric energy that the market can absorb.

The law, of course, is not perfect. Many changes could be suggested, even if every one did not agree on what changes would be desirable. However, after many years, a law has been enacted under which development is rapidly proceeding. It would be folly under such circumstances to permit the law, or the policy which it expresses, to be essentially modified, except after fair trial and convincing evidence of the desirability of change. All those interested should rather unite in working out, within the broad limits of discretion which the Act permits, practices and policies that will hasten the use, under proper conditions, of the vast water power resources of the United States.

HIGH-VOLTAGE POWER TRANSMISSION

By F. W. Peek,* Jr., Esq.

The subject of power transmission will be discussed for the purpose of bringing out limitations imposed by engineering difficulties resulting from the use of high voltages. This will involve a consideration of insulation, regulation, and reliability of operation.

Assuming equal reliability, the transmission voltage must be decided purely from the economic standpoint. Although it is intended to confine this paper principally to the technical problems involved, it will be necessary to consider, to some extent, the economic side. In general, the economical use of the higher voltages will require large concentrated sources of power with a large demand at some distant point; and it seems that in the end this will be the real criterion of the voltage limit and not engineering difficulties of insulation.

During the last twenty years, transmission voltages have increased at the rate of 8 000 volts per year. Engineering has more than kept up with economic demands, and it is not likely that such demands will increase indefinitely at this rate.

In all engineering work, it is necessary to conduct research well in advance of the art. As early as 1910, in anticipation of 220 000-volt transmission, a section of a 250 000-volt line was constructed in Schenectady, N. Y. At that time, the laws of corona, shielding of insulators, etc., were investigated.

After more than ten years, 220 000-volt transmission is about to be realized in practice on two large systems in California. Transmission voltages of more than 1 000 000 volts were investigated in Pittsfield, Mass., during September, 1921, and the investigation has since been carried to 1 500 000 volts. Such high voltages may never be necessary for power transmission, but a study of characteristics over a wide range is of great practical importance.

In order to make the discussion definite and practical, the characteristics of the highest voltage systems at present will be considered first and, later, those of a 1 000 000-volt system will be discussed. It is hoped that a perspective of the whole field will be obtained by considering the two extremes.

As stated previously, high-voltage lines must be able to deliver a large amount of power. It is well, perhaps, to start with a definite idea of what this means. Roughly, 220 000 volts will be required when the power transmitted is 50 000 to 100 000 kw. or more per circuit and the distance is 150 to 300 miles or more. At 1 000 000 volts, economical transmission would probably require at least 2 000 000 kw. per circuit and distances of 1 000 miles or more. The necessity of transmitting a large amount of power per alternating-current circuit at 1 000 000 volts is evinced, for example, by the fact that about 7 000 kv-a. per mile is required to charge a line at 60 cycles. These figures are given only as approximations, the exact figures for any particular case depending on many conditions. However, they do make apparent the

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enormous power and size of electrical units that are necessary for economical transmission at very high voltages, as well as the energy that must be safely controlled during a short circuit.

It is generally desirable to operate each line as a unit with regard to generator and transformers, and this requires quite large units for 220 000-volt transmission. At this voltage, when the power requirement is from 50 000 to 100 000 kw., the space limitation on the railroads has almost been reached for built-up units. At higher voltages, transportation of built-up transformers would not be possible, and it would also be difficult, at present, to obtain material sufficiently large for such transformers.

The apparatus discussed in this paper is the transformer, the switch, the lightning arrester, the transmission-line conductors, the line insulator, and the synchronous condenser. This apparatus will be discussed in order, with regard to a 220 000-volt line.

Transformers.—The transformer which is the connecting link between the high and the low-voltage systems, has undergone some remarkable changes during the last ten years. As the kilovolt-ampere capacity of a system must increase rapidly with the voltage, the energy expended during a short circuit becomes quite large. For a 220-kv. system, it may be 1 000 000 kv-a.

The transformer cores and coils must be able to withstand mechanically the enormous stresses incidental to short circuits on such systems. These stresses may be many tons. The necessity of high mechanical strength has forced the development of a simple structure with circular coils, which has also made a better transformer electrically. The uniform stack of circular coils greatly reduces the abnormal electrical stresses due to arcing grounds.

Transformers are tested to give a factor of safety of 3.46. When the neutral is permanently grounded to the transformer core, the factor of safety is 2.73.

The bushing by which connection is made between the under oil-winding and the air-insulated line wire is an important part of the transformer, and is designed so that it will arc over on the air end before it will puncture. The arc-over voltage of the bushing at operating frequency is three times the line voltage dry and twice the line voltage wet. The apparent factor of safety is, therefore, 5.19 dry and 3.46 wet. The bushing is designed so that its lightning arc-over voltage is several times the factors already mentioned. The transformer has been developed well in advance of the art, and there are no inherent reasons why it cannot be built for much higher transmission voltages, as shown by the fact that the transformers used in the 1 000 000-volt test were of standard design.

Connections.—The method of connecting transformers is of the utmost importance at these high voltages, because if the voltage stresses can be reduced, the reliability of the line will be increased. By connecting the transformers Δ Y and grounding the neutral to the core and ground, the maximum normal frequency stresses are reduced 73 per cent. In case of an arc on a non-grounded system, there are very severe high-frequency oscillations which may cause high voltages to ground and, at the same time, turn-to-turn voltages many times more than normal. These stresses are prac-

tically eliminated when the neutral is grounded, as the arc is then non-oscillatory. If a permanent ground is used, only one transformer bushing is required. This also makes possible a transformer construction which eliminates most of the difficult points to insulate, such as creepage over the ends to the core.

The non-grounded system may be either $\Delta\Delta$ or ΔY. If one line becomes grounded, the normal frequency stresses are increased 73 per cent. The resulting arc is oscillatory and produces enormous stresses between turns in the transformer. The turn-to-turn stresses may sometimes be equal to a line-to-line voltage. If an accidental ground occurs, the abnormal voltages usually cause a break-down on another line or a line-to-line short circuit.

Telephones are inoperative during the arcing. With the ΔY grounded neutral systems, there is no dangerous rise in voltage. If the short-circuit current greatly exceeds the capacity current, the arc is practically non-oscillatory. A single-phase short circuit results immediately and is relieved by the relays.

Neutrals are sometimes grounded through resistance, in order to limit the short-circuit current. If the resistance is high, the characteristics are similar to the non-grounded system, and if it is low, the effect is intermediate.

A scheme has been proposed recently for grounding the neutral of the transformer through a reactance coil adjusted to balance or reduce the capacity current in the arc to zero, the idea being to suppress the arc. This may be desirable sometimes on relatively low-voltage systems. On high-voltage systems, it is difficult to make the adjustment within 1 or 2 amperes or close enough to suppress the arc. In case of a ground of one line, the corona loss may become very great, and it is not possible to balance this loss of current by a reactor. This plan produces, in effect, a non-grounded system with an arc suppressor which may or may not be adjusted sufficiently close to extinguish the arc. Its chief use will probably be on low-voltage systems.

Y–Y connections without delta are undesirable and dangerous and will generally cause telephone trouble. The present practice is tending rapidly toward grounded Y. The direct grounding of the transformer neutral is a necessity for extreme voltages and is assumed in the discussion for voltages at 220 kv. and more.

Switches.—The main purpose of the switch is to operate under the abnormal conditions of short circuit. At the very high voltages, the energy involved in a short circuit will be great. Switches have been developed to take care of short circuits of 1 000 000 kv-a. and more. It is desirable to keep below these values, however, by sectionalizing and by the use of reactance. The switch problem is a difficult one, because of the energy involved in a short circuit.

Line Conductors.—If voltage is applied between the wires of a transmission line and gradually increased, a point is finally reached at which a hissing noise is audible, and, in darkness, a pale glow will be seen to surround the wires. A considerable loss will be noticed, which will increase

rapidly with increasing voltage. This is the familiar corona. The loss increases as the square of the excess voltage above the starting voltage.

In high-voltage transmission, it is important to be able to predict the corona characteristics of a transmission line. Fortunately, this can be done accurately. The voltage at which corona starts, the loss at a given voltage, etc., under any conditions, can be accurately pre-determined. The chief factors affecting corona formation are the diameter of the conductor, the condition of the conductor surface, the spacing, and the barometric pressure. Variation in conductor diameter has a far greater influence than the spacing. The loss increases during storms.

In order to prevent loss at the higher voltages, a conductor of large diameter is necessary. At 220 kv., at sea level, a conductor about 0.95 in. in diameter is required and, at 5 000 ft., the size would be about 1.20 in. For equal conductivity, an aluminum conductor has a diameter about 25% greater than one of copper, and, for this reason, aluminum is sometimes used at high voltages.

A very successful conductor used in California consists of aluminum with a steel core. As the size of a cable is increased, the size of the strands relative to the cable diameter is reduced. As this may have the effect of roughening the surface, and when the corona voltage will not increase as rapidly with increasing diameter for a cable as for a smooth conductor, it is highly desirable to develop a smooth tubular conductor for the very high voltages.

Conductors stranded in various ways have recently been studied. For a given diameter, the maximum voltage is obtained from a smooth tube. For the same diameters, a cable is usually good for only 85% of the voltage of a tube. This ratio is called the irregularity factor. It is of the utmost importance to consider this factor carefully at high voltages.

With conductors about 1 in. in diameter, the 7-strand cable is impracticable. The larger strands become mutilated in manufacture, and the irregularity factor is lower than on cables of a greater number of strands. For new cables, 1 in. in diameter, the irregularity factor was about the same for cables of 19, 37, and 61 strands and varied from 0.80 to 0.85; in new cables, it was generally near 0.80. In some other types of conductors and in mutilated cables, it was lower. The lowest value obtained was approximately 0.70. The irregularity factor probably improves when the points gradually oxidize off with weathering. An aluminum cable of about 1 in. in diameter and several years in operation had a factor of 0.85 or more. The 7-strand cable was not as good, because of the non-uniformity of the cable, due to the stiff strands, and because the stiff strands had become mutilated in manufacture. It is important to have the individual strands of the outer layer free from mutilation and evenly placed. Projecting irregularities should always be avoided. It is also possible to prevent corona loss by using for each conductor three or more wires of the same potential placed closely together. The relative diameters of the two types of conductors at 220 kv. are, as follows: Tubular conductor, 0.77 in.; cable, 0.95 in. It is seen that the line conductor presents no special difficulty at 220 kv. Quite satisfactory results can be

obtained with cables of about 1 in. in diameter and spaced 15 ft. apart. For higher voltages, it is simply a question of getting sufficient diameter with a smooth surface.

The corona loss starts at a given point during each half cycle as the voltage increases, continues over part of the half cycle, and, finally, ends at a given point as the voltage decreases. A varying amount of corona and loss thus occurs during a given part of each voltage wave. The conducting corona, in effect, makes a conductor which periodically varies in diameter. The capacity and loss, therefore, vary during parts of each wave. It follows that if a sine-wave voltage high enough to cause corona loss is applied to conductors, the current cannot follow a sine wave, but must be distorted or contain harmonics. These harmonics are highly undesirable from the standpoint of telephones and otherwise, which is, therefore, another good reason for avoiding corona on transmission lines.

The Line Insulator.—The principal cause of the serious insulator failures has not generally been mysterious high voltages, but poor material and poor mechanical designs of units of the cemented type. The Hewlett insulator, which is free from cement and tight-fitting metal parts, has been free from these troubles. During recent years, the cemented-type unit has been improved, and such troubles have been greatly decreased.

There is another problem, however, which becomes of importance at the higher voltages. When insulators are placed in series in a string, the voltage does not divide evenly. A very high percentage is across the unit nearest the line, whereas even distribution would put 10% on each unit of a 10-unit string. When there are more than 5 units in a string, 20 to 30% of the applied voltage will be on the line unit. It is evident that this becomes of great importance at the higher voltages; for instance, at an operating voltage of 110 kv., the stress on the line unit is 19 kv.; at 220 kv., it is 38 kv.

This uneven voltage distribution may be corrected in a number of ways, one of which is by varying the capacities of the units along the string in proportion to the currents. The spark-over characteristics of a string graded in this manner are bad, as there is a tendency to cascade.

The voltage may be distributed by means of the shield which, in effect, eliminates the capacity to ground in that all the units in the string are the same; it eliminates corona and directs the arc away from the string. By means of the shield and standard, well tried insulators, unit stresses on 220-kv. lines are readily reduced below those at present existing on successful 110-kv. lines. By better distribution of the surface stresses on the individual units, the shield also tends to increase the spark-over voltage when the string becomes dirty.

Although there is considerable room for improvement in the line insulator, it appears that insulator troubles will be less than on the lower voltage lines. It seems time to develop a large unit for use on lines of more than 220 kv.

The insulator or bushing is generally selected so that the wet and dry spark-over voltage at normal frequency is several times the operating voltage. The average ratio of the flash-over voltage of a single string to the operating voltage is 3 dry and 2 wet. Experience has shown these values to be necessary

in order to give good operations. It is interesting that a higher "factor of safety" is required at the lower voltages. This would be expected, as all lines are subjected to the same lightning voltages. From 11 to 14 units will be used on 220-kv. lines. The spark-over voltage decreases with barometric pressure.

It is important to discuss the abnormal conditions that occur on transmission lines before deciding on lightning protection.

Abnormal Conditions on Transmission Lines.—The abnormal voltages that are likely to occur on transmission lines and need be considered in connection with the insulator and transformers are as follows:

1.—Cloud lightning in most cases occurs on the line as an induced stroke. A very steep wave-front voltage travels over the line. At one instant an insulator may be subjected to normal voltage and less than a millionth of a second later to the normal voltage plus or minus the lightning voltage. The lightning characteristics of insulator, bushings, and arrester gaps have been thoroughly investigated by a lightning or impulse generator which the General Electric Company has had for a number of years in the High Voltage Engineering Laboratory at Pittsfield. This lightning or impulse generator is capable of producing voltage waves of a known shape and a duration measured in micro-seconds. The range of this lightning generator has been extended from 800 to about 1 500 kv. The current discharge is measured in hundreds or thousands of amperes. This is far in excess of lightning strokes induced on transmission lines and probably approximates voltages on lines caused by direct lightning strokes. Fortunately, the lightning spark-over voltage of an insulator is always higher than the 60-cycle, spark-over voltage. The lightning spark-over voltage is also not affected by rain or moisture. Observations that have been made by placing gaps of various spacings on transmission lines show that the danger from lightning decreases rapidly with increasing line voltage. In general, it was found that the voltages were of steep wave front. Many discharges took place on the lower voltage gaps. The discharges on the higher voltage gaps were less and less until finally few were found at a needle-gap setting for about 200 kv. This, perhaps, would correspond to about 400 kv. in actual volts. A direct stroke, of course, may be higher. In general, the field tests checked the laboratory tests.

2.—Other transients that are of importance on transmission lines are those caused by arcing grounds on systems with isolated neutrals and those caused by switching. Such disturbances as the General Electric Company has been able to measure are either in the nature of a surge or a highly damped high-frequency oscillation. Such disturbances sometimes amount to double-line voltage, but often cause very little rise in voltage across the line. They may build up, however, to high internal values in inductive apparatus. Fortunately, the grounded neutral system is fast replacing the isolated system. The power arc in the grounded neutral system is practically non-oscillatory. For very long lines at high voltage, however, where the capacity current may approach in value the short-circuit current, arcs may cause oscillations even on the grounded neutral system.

3.—It can be safely stated that undamped high-frequency disturbances practically do not exist on transmission lines. The spark-over voltage for continuous high frequency is lower than the 60-cycle spark-over voltage. The spark-over voltage of an insulator for any transient disturbance that is likely to occur on a transmission line, is always higher than the 60-cycle spark-over voltage. Insulator failures or spark-overs may be caused by excessive voltages, by weakened insulation, or by a combination of the two. Available data indicate that the greater number of insulator failures have not been due primarily to excessive voltages or to "high frequency", but to weakened insulation.

In parts of the United States having a long dry season, dirt collects on the surfaces of the insulators. When this dirt becomes wet with the first rain, dew, or fog, the surface becomes conducting, and the spark-over occurs at a very low value. It is somewhat similar to placing a fuse suddenly across the string. This trouble may be accentuated when there has been mechanical deterioration, and part of the string has become useless. The proportion of failures is small, because it requires a combination of conditions as to dirt, moisture, etc., to establish a conducting path sufficiently long to cause complete spark-over before the dirt is burned off or washed away. Although it is difficult to trace some of these failures to dirt and moisture, it is significant that such failures do not occur where summer rains are frequent. Such rains not only tend to prevent dust, but also wash away the small quantity that collects. The laboratory data given in Table 2 are of interest in connection with dirt formation.

TABLE 2.

Number of units in series.	Spark-Over Kilovolts.				
	Dry clean.	Wet clean.	Dew clean.	Dry dirty.	Dew dirty.
9	537	380	542	537	160

Laboratory tests do not indicate the true wet arc-over on a wet, dirty insulator unless the power supply and testing transformer are large. The laboratory results are generally too high. Ionization of the air around the line conductor, due to incipient corona or other cause, may be dismissed as a probable cause of mysterious spark-over. Tests made in a closed box, where the ionization was much stronger than would be possible in the open air, showed no reduction in spark-over voltage.

Extra High-Voltage Lines.—From the previous discussion, it appears that transmission at 220 kv., or somewhat higher, offers no difficulty. As 220 kv. was about to be realized, it seemed desirable that an investigation be carried to the extreme, in fact beyond any voltage that economic conditions will probably ever demand. At the High-Voltage Engineering Laboratory at Pittsfield in September, 1921, more than 1 000 000 volts was applied to a short section of a 1 000 000-volt line. In this investigation, tests have been made at

1 500-kv., single-phase, 1 000 kv. to ground, and 1 000 kv., three-phase. Briefly, the investigation, as far as it is completed, shows that the laws of corona determined at the lower voltages are followed at the higher voltages. The calculated sphere-gap curve checks the measured one. The needle-gap spark-over is obtained by extending the lower voltage curve. These data are given graphically in Figs. 9, 10, 11, and 12.

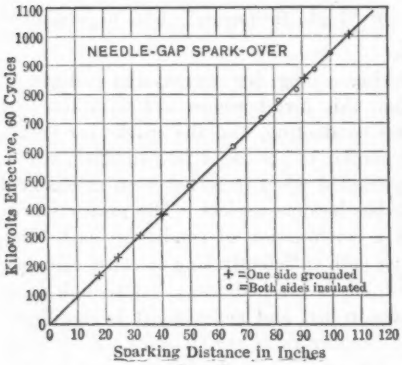


FIG. 9.

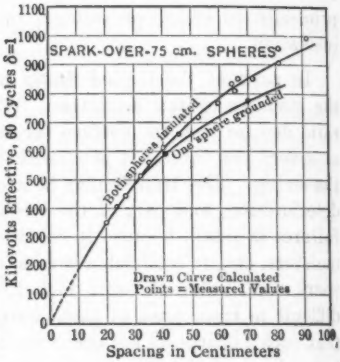


FIG. 10.

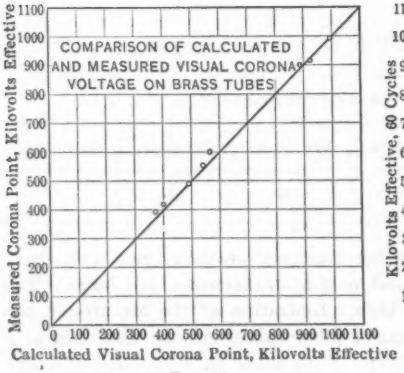


FIG. 11.

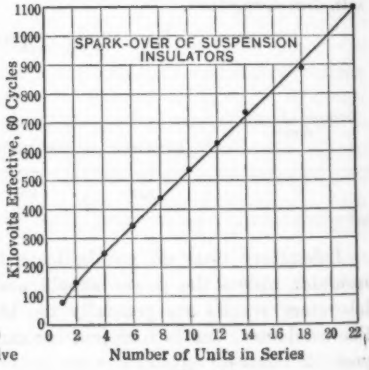


FIG. 12.

In view of this information, it is interesting to discuss the characteristics and practical possibilities of a 1 000 000-volt, transmission line regardless of the fact that present economic conditions do not require such a line. By going to the extreme, a perspective of the whole field will be obtained. The present importance of the problem is to know how to predetermine the characteristics of the higher voltage lines, in order to be ready when conditions demand them. Only the transformer, the transmission conductor, and the line insulator need be considered. The low-voltage apparatus would not differ

from that at present in use, except for the ability to handle, regulate, and control the large amount of energy necessary to make such a line economical. This is the factor that will probably determine the ultimate voltage. The comparison with the 220-kv. line as a background will be of interest.

TABLE 3.—CORONA ON PARALLEL BRASS TUBES, 60-CYCLE, SINGLE-PHASE.

SPACING.		VISUAL CORONA	
In inches.	In centimeters.	Kilovolts, effective, calculated.	Kilovolts, $\delta = 1$, observed.
Diameter, 3.5 in., 8.9 cm.:			
75.5	192	790	730
111.5	283	876	895
147.5	375	915	915
183.5	466	990	990
Diameter, 1.75 in., 4.45 cm.:			
73.7	188	490	490
109.7	279	538	560
145.7	370	568	600
181.7	463	604	675
Diameter, 1.0 in., 2.54 cm.:			
73	185	340	370
109	277	364	380
181	460	402	415

The transformer would not differ radically from the present-day transformer as standard, circular-coil transformers were used in the investigation with more than 1 000 000 volts, single-phase, 1 000 000 volts to ground, and 1 000 000 volts, three-phase.

The calculation of the size of the conductor necessary for such a line is of interest. At sea level and at 30-ft. (9.2-m.) spacing, three-phase, a conductor 5 in. (12.5 cm.) in diameter is necessary at 1 000 kv.

Therefore:

$$e_0 = 21.1\, m_0\, r\, \delta \log_e \frac{S}{r} \text{ kv. to neutral.}$$

$m_0 = 0.90$; r = radius, in centimeters; s = spacing, in centimeters.

$e_0 = 578$ to neutral.

$E_0 = 1\,000$ kv. between lines.

$$p = \frac{244}{\delta}(f + 25)\sqrt{\frac{r}{s}}(e - e_0)^2\, 10^{-5} \text{ kw. per km. per condition.}$$

$$p = 0.08\,(e - 578)^2 \text{ kv. per mile, three-phase.}$$

At 1 000 kv. between lines, the loss is, therefore, zero.

If the voltage is increased 10%, the loss is:

$$p = 0.08\ (635 - 578)^2 = 0.08\ (57)^2 = 260 \text{ kw. per mile}$$

This is quite large. By examining the equation, it will be seen that a peculiarity of the very high-voltage line is that the corona loss increases rap-

idly with a slight increase of the voltage above the critical point. This is so at the higher voltages, because the loss increases as the square of the difference between two voltages. A small increase means a large actual voltage increase.

This is well illustrated by comparing the loss on a 1 000-kv. line and a 220-kv. line for a 10% increase in voltage above the critical value.

For 1 000-kv.:

$$p = 0.08\ (578 - 578)^2 = 0$$

at 10% increase,

$$p = 0.8\ (635 - 578)^2 = 0.1\ (578)^2 = 260 \text{ kw. per mile}$$

For a 220-kv. line:

$$p = 0.05\ (127 - 127)^2 = 0.$$

at 10% increase,

$$p = 0.05\ (130 - 127)^2 = 0.05\ (12)^2 = 7.2 \text{ kw. per mile}$$

Because of the large increase in loss when the voltage is increased, or the critical voltage is lowered, it may be desirable to operate such lines with a larger margin between the critical and operating voltage.

Rain (on the average) lowers the critical voltage 20%; the loss during a storm, therefore, would be:

$$p = 0.08\ (e - 0.8 \times 580)^2 = 0.08\ (116)^2 = 860 \text{ kw. per mile;}$$

a loss of almost 1 000 000 kw. on a 1 000-mile line. For small loss during a storm, it would be necessary to increase the diameter of the conductor to 6.5 in. (16.5 cm.).

In the calculation just given, a fairly smooth tube is assumed. Other forms of conductors are possible. For special conductors or cables, however, it is well to be sure that a decrease in diameter of component parts does not more than offset the gain by the increased total diameter of the complete conductor. The 60-cycle capacity current of such a line would be 4.15 amperes; the kilovolt-amperes per mile would be 7 200 at 60 cycles. The line insulator offers the most difficult problem. The length of the insulator would be from 15 to 20 ft.

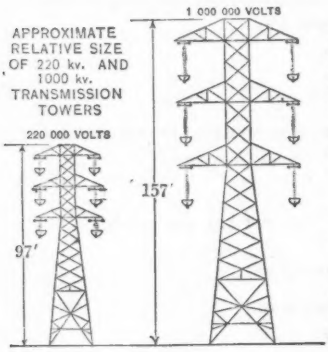


FIG. 13.

A description of a 1 000 000-volt line may now be given: The conductors would be 6.5 in. (16.5 cm.) in diameter, suspended by insulators from 15 to 20 ft. (4.5 to 6 m.) in length, and spaced at least 30 ft. (9.2 m.) apart. A graphical comparison is given in Fig. 13.

Assume that the tubular conductors are made up with a shell equivalent to 1 500 000 cir. mils of copper. At unity power factor, per three-phase circuit, 3 000 000 kw. can be delivered 1 000 miles with 1 000 000 volts at both ends of the line and at an efficiency of 88 per cent. The frequency was taken at 25 cycles in the problem just cited (only for illustrative purposes) to avoid the complication of considering wave transmission. The charging current at no load would be 1 500 amperes, with

1 000 kv. at the receiver end and 700 kv. at the generator end. More complete data are given in Table 4 and Figs. 14 and 15. The control and harmless dissipation of the energy due to an accidental short circuit on the buses of such a system is a problem.

TABLE 4.—CHARACTERISTICS OF 1 000 000-VOLT LINE ON THREE-PHASE CIRCUIT, 25 CYCLES, 1 000 MILES, 6.5-IN. DIAMETER CONDUCTOR, 30-FT. SPACING.

KILOVOLTS.			POWER, IN KILOWATTS.		PERCENTAGE. POWER FACTOR.		Percentage of efficiency.	AMPERES.	
Generator.	Middle.	Received.	Generator.	Received.	Generator.	Received.		Generator.	Received.
Conductor Equivalent to 1 500 000 Cir. Mils, Copper.									
653	908	1 000	97 200	0	+ 5.67	.....	0	1 510	0
760	946	1 000	1 392 000	1 250 000	+ 65.6	100	90	1 610	720
1 000	1 008	1 000	3 270 000	2 870 000	+ 97.8	100	88	1 925	1 650
1 400	1 183	1 000	5 890 000	5 000 000	— 97.2	100	85	2 490	2 880
Conductor Equivalent to 500 000 Cir. Mils, Copper.									
665.0	904	1 000	280 000	0	16.7 lead	.....	0	1 515	0
870.0	991	1 000	1 710 000	1 250 000	68.4 "	100	73	1 667	722
1 000.0	1 052	1 000	2 578 000	1 980 000	83.3 "	100	75.1	1 800	1 145
1 677.0	1 340	1 000	7 575 000	5 000 000	99.99 lag	100	65.4	2 635	2 885

Line Characteristics.—A careful study of the line characteristics becomes of great importance for long lines and high voltages.

To the present time, good regulation has been effected by the use of synchronous motors or condensers at the receiving end of the line. These machines can be made to take leading or lagging current at will by change of field excitation. As leading current has a boosting effect, the drop at full load may be eliminated by making the machine require leading current under load. The tendency for the voltage to rise at no load is opposed by supplying lagging current. By controlling this current properly, there is no difficulty in keeping the receiver voltage practically constant at all loads with a constant voltage of the generator. It has required a synchronous condenser capacity of from 50 to 80% of the generator capacity to accomplish this regulation. It is well to point out, however, that a considerable part of this has been used to correct the low power factor of the load. This excellent method will be used on the two 220-kv. systems already mentioned.

Generators above a certain critical size are necessary in starting very long lines, otherwise the voltage at the sending end will rise to a high value, even when the generators are not excited. This follows because the slight residual voltage of the generators will cause a capacity current to flow to the line which boosts the generator voltage. The current and voltage will continue to rise to a value depending on the size of the generators. It is desirable to have the generators large enough so that one unit can take care of the charging current of the line. Long lines may be started by bringing up the line and synchronous condensers together.

Single, 60-cycle, high-voltage lines are approaching at present a length of 300 miles. As the length increases a new problem is presented; the line tends to become unstable. The length at which instability occurs depends on the frequency and is due to a peculiar resonant condition that occurs at a critical length. The critical length is approximately 750 miles at 60 cycles and 1 800 miles at 25 cycles. In brief, this condition is due to the fact that it requires time for the electric energy to travel over a wire. With the distances just mentioned, the first wave travels to the end of the line and back to the generator in time to add to the second wave, and so on. These critical distances are called the quarter-wave lengths for 25 and 60 cycles, respectively. Instability will begin to be noticed at distances considerably less than these. At no load and very low generator voltage, the voltage at the far end of such a line would reach high values, being limited, in fact, only by the losses.

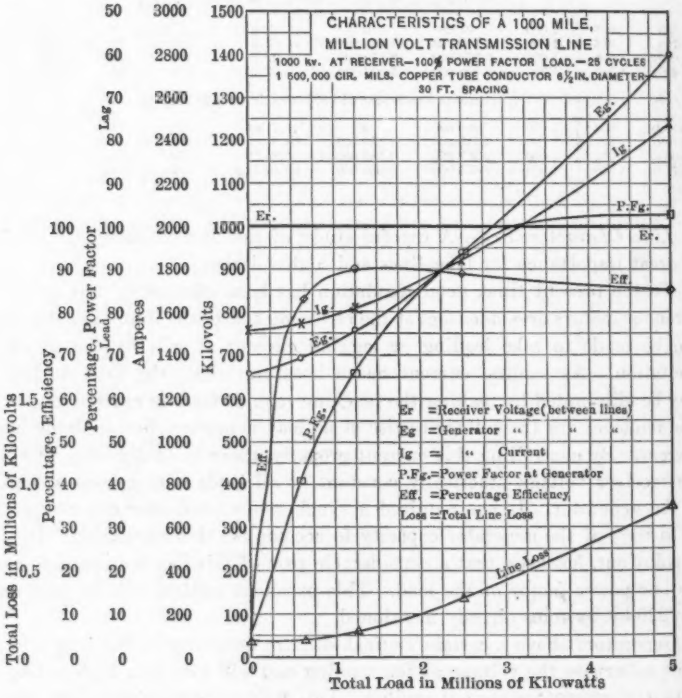


FIG. 14.

It is possible to make use of the peculiar characteristics of such lines. For instance, if approximately constant current is held at the generator, practically constant voltage results at the receiver. Under load, however, the voltage at the middle of the line may be quite high. The characteristics are shown in Fig. 16. At double the distance the half-wave line results. Its char-

acteristics are better than the quarter-wave line. In both, however, there are practical difficulties. The voltage may be high at intermediate points under load and go to very high values at no load. Intermediate taps are also difficult or impossible to make.

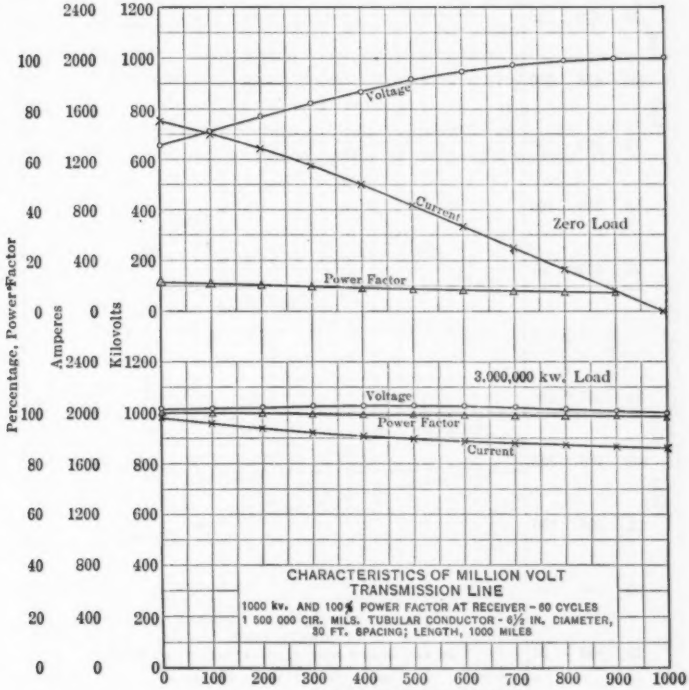


FIG. 15.

The simplicity and flexibility of the synchronous condenser regulated line are lost in quarter and half-wave transmission. This has suggested the extension of the present system by distributing synchronous condensers along the line at intervals considerably less than the quarter-wave length, and controlling the phase relation between voltage and current at these points. Each section is, in effect, an independent short line.

There is no untried principle in this, as synchronous machines have been used and distributed over power systems for years. The plan has been admirably discussed in a paper by F. G. Baum, M. Am. Soc. C. E.* This method holds more promise than any of the other schemes considered.

Direct current offers some admirable characteristics for transmission to great distances, which are worthy of mention. The present difficulties come in

* "Regulation and Insulation for Large Power Long-Distance Transmission", *Journal*, A. I. E. E., June, 1921.

obtaining high direct-current voltages and, later, in stepping down for distribution. For a given distance in air or oil, the direct-current voltage required to cause break-down is about 41% higher than the alternating-current voltage. The advantage of direct current is about 2 to 1 for solid insulations.

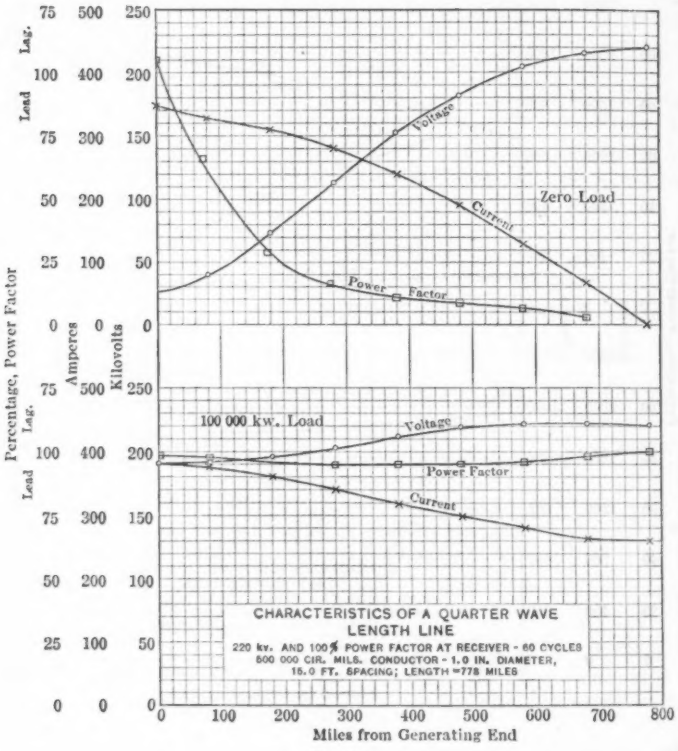


FIG. 16.

It may be of interest to mention that, for every alternating-current line, there is a certain critical current at unity power factor load where the characteristics of the line are the same as for a direct-current line.

Conclusions.—

1.—The maximum voltage used in transmission will not be imposed by engineering difficulties of insulation, but by available supplies of energy. Although it is not probable that economic conditions will require 1 000 kv. for power transmission, the characteristics of such a system have been discussed in order to get a perspective of the whole field. Very high voltages are not economical unless the power is enormous.

2.—Transmission at 220 kv. will generally not be economical unless the power transmitted is of 50 000 to 100 000 kw. per circuit and the distances from

150 to 300 miles or more; at 1 000 kv., economical transmission would probably require more than 2 000 000 kw. per circuit and distances of 1 000 miles, or more. The limitations imposed by the large size of the apparatus units are probably more difficult than those imposed by insulation. A great problem is safe control of enormous amounts of energy during a short circuit.

3.—Considerably higher voltages could be used at present if economic conditions required it. The transformer has been developed far in advance of the art; the line conductor would offer no difficulty; although considerable development is desirable in the line insulator, it does not appear to have serious limitations.

4.—For very long-distance transmission, it appears that stability and good regulation will be secured by distributing synchronous condensers at intervals along the line.

SOCIAL AND ECONOMIC ASPECTS OF HYDRO-ELECTRIC POWER

By Charles D. Marx,* Past-President, Am. Soc. C. E.

All engineers are so firmly convinced of the important rôle which hydro-electric power is playing and will continue to play as a factor in human development, that they do not take time to marshal the facts in support of this statement. An effort to do this, if only in part, will be attempted.

A California writer has said:

"What is civilization? It's where you stand on a street corner and wait for ten street cars, four hundred automobiles, fifty motor-cycles, twenty-five bikes, a dozen motor trucks, a few fire engines, the police patrol, a hurry-up ambulance, and a funeral, to pass before you dare make a try for the opposite corner."

If that is a correct definition of modern civilization, and indicative of its benefits to mankind, the speaker would not blame Mahatma Ghandi for the strenuous efforts he is making, not only to prevent the spread of Western civilization in India, but also to destroy it, through his policy of non-co-operation.

Modern civilization depends on the economic development, transmission and distribution, and the intelligent utilization of the sources of energy for the creation of wealth. Civilizations have existed in the past, and the monuments remaining in India, Egypt, Mexico, Peru, and, later, in Greece and Rome, bear testimony to the ability of our predecessors to utilize some of the sources of energy for the creation of wealth, but also that this energy, largely man power, was not economically employed and the wealth created was not intelligently utilized, owing to its unequal distribution.

Consider, briefly, the different kinds of energy available for the creation of wealth. There is first the energy exerted directly by man, which was the kind in most general use for the development of ancient civilizations. The amount developed, as shown by the accumulated wealth, depended on conditions of food, climate, and soil, and the ability to overcome physical obstacles. It was large in Asia, Africa, and America, the seats of the most ancient civilizations. The wealth so created, however, was distributed very unevenly, and human energy was often misdirected.

We are told that, "* * * two thousand men were occupied for three years in carrying a single stone from Elephantine to Sais", and that, "to build one of the Pyramids required the labor of 360 000 men for twenty years." Surely this does not represent the economic development of one of the sources of energy.

Modern civilization will continue to depend, at least in part, on the direct utilization of human energy, and the amount available will depend on the population and its physical condition, the latter, in turn, being determined largely by the conditions of living. As bearing on this question of future available man power in the United States, one cannot do better than read

* Prof., Civ. Eng., Leland Stanford, Jr., Univ.: Cons. Engr., Stanford University, Calif.

the article* by Frederic C. Howe, entitled, "Has the Westward Tide of Peoples Come to an End?" He believes that:

"The result of these conditions [from the restrictions on immigration now in force] may be a reduction in the productive capacity of the nation. It will certainly reduce the number of persons occupied in manual pursuits, and especially in the elementary pursuits that are now manned by the raw material from Europe, which, for a generation, has been cheaper than the building of machines to do man's work."

If this forecast is correct—and Mr. Howe marshals the facts in support of his conclusion—the importance of conserving all other sources of energy for the creation of wealth will be recognized.

The speaker stresses here, and in his original definition, the creation of wealth as the essential factor in the development of civilization, because the creation of wealth implies a surplus, and it is only when there is a surplus of wealth—a quantity greater than is needed to meet the physical needs of peoples from day to day—that there can be had that leisure for the development of the thinking power of man on which depends both his material well-being and spiritual growth.

The development of the other sources of energy call, in an increasing measure, for the application of human knowledge, in order that a maximum of economic development of these sources of energy may be obtained. "We simply must have cheaper fuel—cheaper energy of all sorts—or every one of us is going to have a harder time making a decent living", says President Little, of the Institute of Chemical Engineers. He claims we must:

"(1) Mine and burn coal more economically; (2) use artificial anthracite in our homes; (3) get more heat out of gas; (4) find a new and better motor fuel; (5) develop more water power."

He shows also that there are other untapped sources of heat and power among which are:

"(1) The immense energy of the sun's rays; (2) the energy of the earth's rotation; (3) the power of the rising and falling ocean tides; (4) the steam of volcanoes; (5) the vast forces stored in the atom."

Among the sources of energy one must distinguish between those the utilization of which spells a destruction of the capital of energy, and those which may be said to be self-renewing. The continuity of civilization depends on the economic utilization and development of the latter classes. Of these, the one most developed at present is the energy obtained from falling water. The others depend on the growing knowledge and skill in making available the other potential sources of energy. Progress in the arts and sciences requires time, and should this progress be slower than the consumption of energy from sources which connote a destruction of the sources of energy, civilization would not only come to a standstill, but would also face destruction. It is because of the self-renewing energy of water that hydro-electric development becomes a factor of such supreme importance for the civilization, not only of this country, but also of other countries where only a beginning of its use is taking place.

* *Scribner's Magazine*, September, 1922, p. 362.

The speaker has briefly referred to India and the efforts being made there to arrest the progress of modern civilization and to turn back to a type for which the claims are made that it is better and more conducive to the happiness of the human race than that now being developed. White Khaddar hand-woven cloth, made of hand-spun yarn, is to take the place of modern cloth; railroads, telegraphs, modern methods of transportation, disseminating knowledge, increasing the productivity of the soil, sanitation, increasing the length of life, and bettering the living conditions of the people, are to be scrapped, for the purpose of holding a civilization that has been static for thousands of years—a civilization of a type that spells poverty for the many and the degradation of humanity. This is a doctrine preached to hundreds of thousands in India, by a man who arrives to do his preaching in a Ford motor car!

However, the prophets of modern civilization in India, the engineers, are busily engaged, through the development of hydraulic power, in bringing about a change in this civilization—a change for the better. The late Mr. Robert Batson Joyner, in his paper on "The Tata Hydro-Electric Power-Supply Works, Bombay,"* describes the improvement of Indian conditions by water power, as follows:

"The work now described shows that, given head enough, it is financially possible at certain favorable places, to store water to provide power during about three-fourths of the year when the rivers are dry, and India should now be able to supply itself with many articles which hitherto have had to be imported; because, though so much of the raw materials is grown in the country, and labor is cheap, power from coal is much too expensive in most parts of India to enable it to compete with other countries, especially as the cost of ocean freight has been so much reduced. The cheap water-power now shown to be producible at favorable sites should greatly promote the prosperity of the country, not only by the establishment of large factories, but also by the use of the stored water, after giving up its power, to help to ward off famine. It should also ensure the growth of the raw materials required, provide transport for these materials to, and for finished products from, the factories, produce the fertilizers needed, and supply food and drink as well as light to the workers and others."

One of Ghandi's cries is "Swadeshi", which might be translated, "India for the Indians." This is a legitimate desire for any nation, and the British hydraulic engineers, with the hearty support of the much maligned British Government, are helping the Indian people toward a realization of this dream. The opposition of some of the Indian people toward Western civilization, and the charges brought against it, however, should cause no wonder. One need not go back farther than 1770, when there was a widespread feeling that civilization was not all that it might be; that civilized man fell short of the ideal. Mr. J. C. Squire, in the *London Observer*, reviewing Professor Tinker's book, "Nature's Simple Plan, a Phase of Radical Thought in the Mid-Eighteenth Century", states, "It was very generally held—even to-day the dispute is a familiar one in school debating societies—that the savage was better off than the civilized man; happier, stronger, more genuinely poetical. The noble savage was one ideal." Even, to-day, the tales of the South Sea Islands might lead people to think that happiness is in direct pro-

* *Minutes of Proceedings*, Inst. C. E., Vol. CCVII (1918–19, Pt. 1), p. 53.

portion to our ignorance of the powers of Nature, and our ability to utilize them for the benefit of Man.

These views are often held by those who are benefiting most by what the engineer has done in unlocking the stores of energy and making them the servants of man. Literary scribblers, in steam-heated dwellings, with all the modern conveniences at their call by the pressing of a button, and ignorant of the high mental development back of the button which places these conveniences at their disposal, inveigh against modern civilization and its materialistic tendencies. They lose sight of the fact that it is the creation of surplus wealth by the utilization of Nature's energy which makes their existence possible and has raised them from the condition of savages to that of civilized men.

Another interesting development of hydraulic power, which may change the conditions of civilization in Palestine, is that which contemplates the utilization of the waters of the Jordan and other rivers in that country for the generation and distribution of electric power.* The engineer who is urging this development claims that, as regards Palestine, the British mandate, although important, is less so than the hydro-electric development for bringing about peace and prosperity there. "What the mandate is to mean," he says, "depends on the future of hydro-electric power. Make that cheap and accessible and the mandate can prove itself by the economic prosperity it will bring." Mr. Rutenberg insists that the evolution of the scheme is one of the real ways to remove the existing barriers between the Jews and the Arabs. "Offer them the chance of continuous labor," he says, "let them see the obvious economic advantages of the scheme, and they will cease to debate idle differences of principle that have no substantial reality." If hydro-electric power development can successfully wipe out hatred of race and creed, as claimed by Mr. Rutenberg, might it not be desirable to push its development wherever possible in other centers of strife? "La Houille Blanche", the white coal of France, if utilized, might diminish that country's longing for the black coal of the Ruhr, and if there are water-power possibilities in Ireland, peace might even come to that distracted country.

This discussion so far has been confined to the presentation of the economic and social aspects of hydro-electric power, in reference to nations with a civilization differing from our own. The effects of such development are likely to be far-reaching and beneficial. The speaker has merely advanced a claim, but a claim which he thinks can be substantiated as one turns to an investigation of what the development of hydro-electric power has meant in the United States.

Probably no man is better qualified to speak of the effect of electrical engineering on modern civilization than Mr. Charles P. Steinmetz. In an address† before the Franklin Institute in 1914, he stated that:

"The necessities of civilized life consist of two groups: materials, and energy. Our transportation system takes care of materials, but cannot deal

* An article on the technical features of the proposed development has been published in *Engineering News-Record*, July 22, 1922.

† "Effect of Electrical Engineering on Modern Industry", *Journal*, Franklin Inst., Vol. 177 (1914). p. 115.

with the supply of energy, and the failure of an efficient energy supply has been and still is the most serious handicap which retards the advance of civilization. The transportation system could deal with the energy supply only in an indirect manner, by the supply of materials as carriers of energy, and when our railroads carry coal, it is not the material which we need, but the energy which it carries. But this energy is available only to a very limited extent, as heat, and as mechanical power in big steam units: most of the demands of civilized life could not be satisfied by it. In any country village far away from the centers of civilization we have no difficulty to have delivered to us any material produced anywhere in the world; but even in the centers of civilization we could not get the energy to run a sewing-machine or drive a fan without electric power. Thus, just as our steam railways and express companies take care of the transportation and distribution of materials, so civilization requires a system of transmission and distribution of energy * * *."

The speaker stated in the beginning of his paper that civilization depends on the economic development, transmission, and distribution of energy, and, furthermore, he called attention to the fact that the store of such energy must not be diminished, if civilization is to endure. It is because of the fulfillment of this last condition, that hydro-electric power plays the important rôle it does in modern civilization.

The economic development and utilization of hydro-electric energy is conditioned on a number of factors. The cost of production, aside from the cost of construction of the plant, is a function of its rate of use, as thus far the development of the storage battery has not progressed sufficiently to be considered for the storage of electrical energy on a large scale. Hence, it is desirable for the most efficient utilization of electrical energy, that there be created a uniform demand or load curve. F. G. Baum, M. Am. Soc. C. E., has shown in an interesting paper,* how electric transmission provides the elastic medium for exchanging any kind of mechanical power to any other form of mechanical power at some point in the system; and also how the inter-connection of a hundred small plants by an electric transmission system with a large power demand, inevitably leads, in order to obtain economy in operation, to the installation of a few large units to supply the entire system. The interconnection of these large power units, in turn, will permit of an economic utilization of all the power developed, as was shown clearly by the results obtained during the World War, when the people of California faced a power shortage.

Under modern industrial conditions, electric power has a large rôle to play, not only in conserving the energy supply of the nation, but in increasing the amount of man-power energy available by increasing the food supply. All the free land of America is gone. Intensive cultivation must take the place of wastefulness in production which has characterized American farming; but intensive cultivation in the absence of adequate man power means that many things now being done by hand must be done by machines. Then, too, there is a steady disinclination on the part of the people of this country to do ordinary manual work. Electrically operated machines in many industries will lead to a substitution for this man power; even in domestic science, one

* "The Economic Value of Electric Transmission", *Journal of Electricity, Power, and Gas*, January, 1914.

finds the great rôle which the application of electricity is now playing and will continue to play, with increasing importance. The housewife, for her domestic duties, cannot hire a human servant as willing as electricity, with its constant novel adaptation to her needs. Electricity solves in a large measure the vexing problem of domestic help, and brings about a far-reaching change in our daily life. The servant class, as a class, is likely to disappear, and who will say that this is not a social change for the better?

Not only in the food-producing field of agriculture and the food-consuming, energy-producing home, must one look forward to added conservation of human energy. All industry demands the replacement of man power by machinery, and hydro-electric energy is being utilized, both as a conservator of energy, from other sources, and as a converter of minerals and the constituents of the air into fertilizers and other products for the use of man. The field of application is almost limitless, and as it steadily widens, as the intelligence of man, with its constantly developing inventive genius, succeeds in making not one blade of grass grow where none grew before, but many thousands of them, the burden and drudgery of mankind for gaining a daily living will be lightened.

Modern civilization depends on the economic development, transmission and distribution, and the intelligent utilization of the sources of energy for the creation of wealth. The speaker has stated advisedly, the intelligent utilization of the sources of energy for the creation of wealth, because he believes that this intelligent utilization carries with it the intelligent distribution of the wealth so created. Therefore, he believes that the development of hydro-electric power will help solve this vexing problem.

As conditions of living on the farm are made more attractive, as drudgery grows less, and loneliness disappears, as the opportunity of being an owner of land, and not a tenant farmer, becomes greater, as the result of intelligent colonization schemes, such as have been carried out at Durham and Delhi, in California, under the wise leadership of Elwood Mead, M. Am. Soc. C. E.; as such changes are brought about—and they are made possible largely by the use of hydro-electric energy—the creation of wealth is furthered, and its intelligent distribution is accomplished.

Dr. Mead reports*:

"Two canny Scotch farmers came to look over these settlements. One of them went to the back door of one of the farm laborer's homes, and the story of the wife of that farm laborer, will be his text in Scotland. Telling one of his impressions he said, 'I want to say, I have visited all the agricultural countries of Europe. I have visited a great part of the world, but I am going home to tell our people that the finest thing in rural democracy that I have ever seen, are the farm laborers' homes in that settlement at Durham.'"

In California, at least 1 500 000 acres are under irrigation by water electrically pumped, and 83% of the dwellings are wired for electricity. The domestic heating load grows. The use of hydro-electric power for industrial purposes shows a steady increase, and this means that other sources of energy are being conserved and wealth produced without a corresponding loss of

* *Transactions*, Commonwealth Club, November, 1921.

energy. It proves that for that part of the United States—and what is true there holds true elsewhere—the conservation of energy made possible by the use of hydro-electric power, tends to insure the permanency of a civilization which, although perhaps not ideal, has after all changed the conditions of living of millions of people for the better.

The speaker believes he has shown, if only imperfectly, that in the social and economic changes which are taking place, and by which the state of civilization is measured, hydraulic power has played an important rôle; and that it will play an increasingly important one, he firmly believes.

AERIAL PHOTOGRAPHY AS AN AID IN MAP MAKING, WITH SPECIAL REFERENCE TO WATER POWER SURVEYS

By Gerard H. Matthes,* M. Am. Soc. C. E.

The purpose of this paper is to bring to the attention of the Profession a method of utilizing aerial photographs in the preparation of maps, which has been used with considerable success in a recent survey of the Tennessee River. The method appears to be well adapted for use in water power surveys, irrigation surveys, city planning, and the location of highways. It utilizes to the fullest possible extent the information furnished by the photographs, and eliminates the necessity of providing triangulation or other costly forms of ground control. In this respect, it departs not only from ordinary survey methods, but also from most of the systems of aerial mapping evolved to the present time.

As a result of the experimental work with aerial photography, that has been done in the United States and abroad, two distinct products have been developed: The mosaic, made by joining a number of photographs into what purports to be a continuous photographic reproduction of a part of the earth's surface; and the hand-drawn map compiled from aerial photographs, which has the appearance of an ordinary map. This paper is devoted mainly to a consideration of the hand-drawn map, including both the plane and the topographic map. A brief description of the mosaic has been added for the purpose of making clear its advantages and limitations. No practical method has yet been evolved for showing the relief by means of contours by direct photographic or stereoscopic process. Some progress in that respect has been made in France and Germany, but in this country it is still necessary to supplement the contours by surveying. This is best accomplished by the plane-table method, either by drawing the contours on skeleton maps prepared from the photographs, or, by drawing the contours directly on the photographs. The success that the speaker has had with the latter method inclines him to the belief that eventually it may find wide application.

The Mosaic

The mosaic gained its popularity during the World War when quick work in locating enemy positions, rather than the production of maps, was essential. The usual method of constructing a mosaic is by fitting and pasting to a suitable backing, parts of adjoining photographs. This is done by selecting the clearest and best parts of the photographs, usually their centers because these are freest from distortions, and discarding the remainder in such a manner as to leave a feathered edge of irregular contour. This feathered edge avoids the appearance of distinct matching lines in the finished product, and, when wetted, it may be stretched and squeezed a certain amount, which is a great aid in securing perfect matching of contiguous details. In expert hands, the continuity that can thus be obtained, without revealing unseemly edges or unavoidable discrepancies, is quite remarkable.

* U. S. Asst. Engr., U. S. Engr. Office, Chattanooga, Tenn.

In a sense, each component photograph of a mosaic is a miniature map, that is, assuming it to have been taken with the focal plane of the camera held truly parallel to the ground surface. Unfortunately, with the best equipment now available, it is impossible to take all the photographs in one flight, not to mention several adjacent flights, at the same altitude, or with the focal plane of the camera always parallel to the ground surface at the time of exposure. Nor is the surface of the ground a plane surface. From these sources of error result the following: (*a*) differences in the scale of the photographs due to the varying altitude of the aeroplane; (*b*) distortions due to tilting of the camera at the instant of exposure; and (*c*) displacements of objects from their true positions, due to inequalities in the elevations of the land.

However slight these errors may be, and they usually are slight, their sum total makes the construction of a first-class mosaic quite difficult, and if great care is not exercised, only indifferent results can be expected. Unless some form of control is available, that will serve as a check on the position of objects at frequent intervals, there is no assurance that the scale of the mosaic can be made dependable. In the case of mosaics of cities, such a check is usually obtainable by comparison with existing maps, but where no accurate map is available, a ground control, by triangulation or other survey methods, must be established and this greatly increases the cost. Even with the best of control, it is at times difficult to avoid unsightly offsets in mosaics, due to distorted parts not matching properly.

The inaccuracies that are inherent in ordinary mosaics do not operate to debar them from many practical uses. Thus, they are excellent as reconnaissance maps, and for other purposes that do not require the accurate scaling of distances or the close measuring of areas. They have been found of value in the study of traffic problems in large cities; for ascertaining the location of enemy positions in time of war; in studying railroad and dock terminal facilities; in real estate development; and in city planning. Mosaics possess the merit of being obtained usually at a small outlay of cost and time. Until greatly improved methods are evolved for taking photographs from the air, the field of usefulness of the mosaic, to the engineer, is likely to remain limited.

Hand-Drawn Maps

Under this subject will be considered maps that are compiled by transferring information from aerial photographs directly to tracing linen, or to paper. The special method advocated herein is not applicable to the mapping of large areas on relatively small scales, such as the standard base map of the U. S. Geological Survey, etc., for the reason that the primary requirements of maps of that class are to show the objects in their correct positions with respect to meridians and parallels or grid lines, and to take due account of the sphericity of the earth. It is obvious that the requirements of such maps cannot be attained through the use of photographs, except by providing a rigid system of triangulation, that will effectually place each particle of information derived from the photographs in its proper geographic position.

The new method appears to be especially suited to the class of maps required by the practising engineer in the planning of engineering works. However heterogeneous in aspect and purpose the maps required for engineering purposes may be, they differ as a class from geodetically accurate maps in three important particulars: (*a*) they are not units in a comprehensive mapping scheme; (*b*) the location of objects with respect to meridians and parallels is not essential; and, (*c*) the limit of permissible error is fixed by considerations of a practical rather than of a cartographic nature. There is an unlimited field for the application of aerial photography in the preparation of engineers' maps, but it has not received the attention that has been accorded it in the production of geodetically accurate maps.

The experience gained on the survey of the Tennessee River demonstrates that, within certain limits, excellent results can be obtained without resorting to any form of horizontal control other than that derived from the photographs themselves.

It is not the province of this paper to give a detailed description of the process of constructing aerial photographic maps used by the Army Air Service, as descriptions thereof are available.* An outline of the process is here given in order to enable the reader to judge in what respect it differs from the method about to be described.

The method used by the Air Service consists in covering the region systematically with overlapping courses of photographs and in determining the correct position of the individual photographs, on tracing cloth or celluloid, by the Bagley method of orientation a description of which is given subsequently. Adjustment is then made so as to bring the position of the photographs in accord with the horizontal control. After this is accomplished, a skeleton map is compiled on sheets suitable for use on a plane-table by transferring the major details from the photographs by the use of a pantograph. These sheets are then ready for the topographer to take into the field. All this pre-supposes, however, that the necessary spirit-level and triangulation control has been provided.

The Survey of the Tennessee River†

The primary objects of this survey and investigation were to ascertain the possibilities of constructing dams across the Tennessee River and its principal tributaries for the joint development of water power and navigation. Secondary objects were the securing of data for use in studies of flood control and the development of the mineral resources of the region, in so far as these subjects had a bearing on power and navigation. It was recognized from the beginning that an accurate and detailed map would be needed, that should furnish all necessary information for making preliminary estimates of cost of structures and land damages; also, that such a map should be of sufficient scope to show the territory adjacent to the river for a distance of a mile or

* "The Use of Aerial Photographs in Topographic Mapping," *Information Circular No. 184*, published by the Chief of Engineers, U. S. Army, 1921.

† Ordered by the River and Harbor Act of June 5, 1920.

more, in order that intelligent study might be given to the location of industrial sites with reference to flood levels, water and rail transportation, power-transmission lines, highways, and towns. Existing maps did not furnish this information, nor was it possible by combining all available sources of information to compile a dependable map fulfilling these requirements. This deficiency arose from the fact that the quadrangle maps of the U. S. Geological Survey of this region were inadequate, being on a scale of 1:125 000, the surveys having been made nearly 40 years ago, whereas the river charts prepared by the U. S. Engineer Department, although drawn to a large scale, supplied no information outside the river banks.

The new map was undertaken on a field scale of 1:15 000 (about 4 in. equal 1 mile) and a scale of publication of 1:20 000. It was to show all watercourses, the configuration of river banks, islands, and sloughs; all railroads, highways and roads, including private roads in the vicinity of the river; all isolated buildings of importance; all the more important property lines as indicated by fences, and the classification of lands, as follows: (*a*) high priced, cultivated lands; (*b*) hilly farm lands; and (*c*) wooded, rocky, or gullied lands. The relief was also to be shown by 5-ft. contours in the valley bottoms and 10-ft. contours on the hillsides, thus making it possible to compute the areas of lands that would be overflowed for any reasonable height of dam that might be planned in any given locality. The contours were located for a distance of at least 60 ft. above extreme low water in the Tennessee River.

The main object in using aerial photography was to expedite this undertaking and thereby to save cost. It was estimated that about 2 000 sq. miles in the form of a narrow ribbon covering many hundreds of miles of river valley would have to be mapped, and the funds available were inadequate for securing, by ordinary survey methods, a topographic map of the desired scope and accuracy.

There was little precedent that would serve as a guide. A number of topographic maps, however, had been made with the aid of aerial photographs, and a limited supply of literature was available on that general subject, but hardly any of it seemed to fit the particular case in hand.

The co-operation of the Army Air Service was enlisted for taking the photographs, and this resulted in the photographing of the valley from Knoxville to Chattanooga, Tenn., during the summer of 1921. In October of that year, at the inception of mapping operations, a method was tried out involving the sketching of contours directly on aerial photographs, and this gave promise of good results from the beginning. It consisted of taking stadia topography by the plane-table method, and platting the points, as fast as they were located, on the photographs. The contours were then sketched, the photographs sent to the office, and the information on them transferred to tracing cloth into final map form.

Although contour sketching on photographs had been done to a considerable extent in France and Germany, and experimentally in the United States by the U. S. Army and the U. S. Geological Survey, no instance is known of

such work having been done methodically over any large area in this country, previous to its adoption in the survey of the Tennessee River. The field work on this survey was conducted for eight consecutive months, when the funds became exhausted. During that time, the survey developed from an experiment into a well grounded system of map making, the results of which surpass the older methods of surveying and mapping in the degree of faithfulness and completeness of detail obtained and by the saving in time and cost. The following is a description of the various phases of the work.

Aerial Operations.—The photographing was done by two officers of the Army Air Service from a De Haviland plane equipped with a camera of the K-1 type, which had a single lens of 10-in. focal length. It was necessary to fly the plane at an altitude of 12 500 ft., in order to obtain photographs on a scale of 1:15 000. A variation of 125 ft. in the altitude of the plane affected the scale of the photographs exactly 1 per cent. The photographs measured about 7 by 8¾ in. In terms of distances on the ground, these dimensions corresponded to 1.66 by 2.07 miles.

The photographic equipment, although not of the latest type, was complete, and included the following accessories: An adjustable gimbal-like support for the camera, which permitted it to be leveled quickly, and to be rotated; a Venturi vacuum tube which, through air suction, served to hold the film flat against the focal plane plate at the instant of exposure; an electric battery for operating the automatic mechanism; a speed-control device for regulating the interval between successive exposures; a view finder which enabled the photographer to determine the proper setting of the speed-control device and the amount the camera must be rotated to compensate for "crabbing" of the aeroplane due to its position at an angle with the true direction of flight.

The normal velocity of the plane while the photographs were being taken was 85 miles per hour. As the photographs measured 1.66 miles in the direction of flight, and an overlap of 50% was desired, it was necessary to make exposures about ⅘ mile apart, or at intervals of 35 sec. in still air. In case of wind, proper allowance had to be made, in order to maintain the required lap.

The films came in rolls and were 24 cm. wide by about 75 ft. long, and contained about 100 exposures. As it was not practicable, with the equipment then used, to change films while flying, the work had to be planned to conserve film and time to the best advantage. This was important, as in Tennessee the number of days when atmospheric conditions are favorable for aerial photographic work do not usually average more than 8 per month.

For each day's work, which usually consisted of 2 to 3 hours of flying, the pilot was furnished with a section of map from a quadrangle map of the U. S. Geological Survey, on which were shown, by colored lines, the flights to be made. The work was laid out in parallel and overlapping courses, averaging about 10 to 15 miles in length. Sixty-eight courses were covered, and more than 1 000 photographs were taken of the valley from the head of the Tennessee, near Knoxville, to Chattanooga. The total area covered by photographs was 675 sq. miles, including 188 lin. miles of the Tennessee River, and 20

miles of the Hiwassee River, from the mouth of the latter to Charleston, Tenn. Along the main river, the belt of photographs varied in width from 3 to 9 miles, with an average of 5 miles. The greater widths were made necessary by river meanders and because of having to include certain tributary valleys.

The entire area was covered from one landing field in East Chattanooga. It was common for the aviators to fly to Knoxville, a straight-line distance of about 100 miles, make from 50 to 75 exposures, and, without alighting, return to Chattanooga in time for luncheon.

Because of weather conditions, seven weeks were required in which to complete the aerial work, photographs having been taken on only eleven days of that period. These figures show that, in the southeastern part of the United States, the cost of aerial photographic work is a function of climate rather

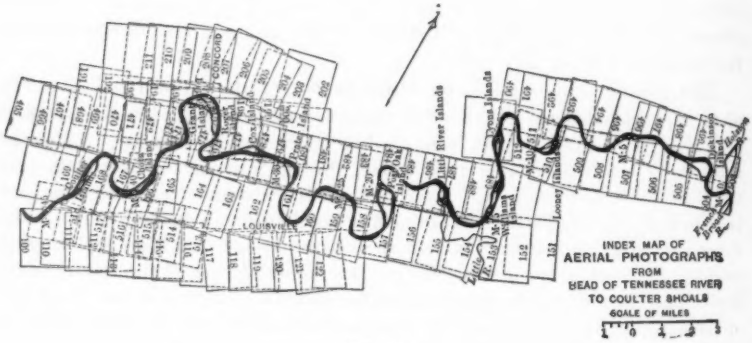


FIG. 17.

than of the size of area covered. Had this photographic work been done in the arid regions of the West, two weeks probably would have been required.

It was intended to have the photographs within any course overlap 40 to 50% in the direction of flight and about 15% between adjacent courses. In some instances, the lap obtained was much less, as may be seen by reference to Fig. 17 which shows a part of the index map prepared for the survey. This index map shows the relative positions of the individual photographs, for general reference, without any attempt at precision. Fig. 17 is not to be taken as representative of the aerial work done on the Tennessee River, but it was selected because of its shortcomings which make it more instructive than a more perfect example.

Thus, the two long courses, numbered from 151 to 169 and from 465 to 489, show a scant overlap which was the cause of much extra labor in assembling the final map. Again, the short course, numbered 202 to 211, gives evidence of the aeroplane having had trouble, because of some local atmosphere condition, and, as a result, there is insufficient lap on the course to the south, and at one place there is no lap whatever. In some instances, it was necessary to fly supplementary courses in order to close hiatuses, as, for instance, near

Mile 45 on the Tennessee River, where the course numbered 513 to 518 was needed to cover the gap left between the courses numbered 109 to 122 and 151 to 169. As the developing and printing was done at Langley Field, Va., it was sometimes two weeks or more before such troubles became known. The disadvantage of having the dark-room work done at a distance was felt also in other ways, and impressed those in charge with the necessity of the provision, in future work of this kind, for having the developing and printing done locally.

An example of adequate lap is shown by the courses at the extreme right in Fig. 17, both as regards lap within courses as well as between them. Unfortunately, there is hardly any lap between the lower of these two courses and those immediately to the southwest. This made map construction at that point impracticable by the method adopted. Fortunately, it was possible by reference to the old river survey charts to secure proper continuity of the photographic map. The extent to which these early charts were utilized in checking the aerial photographic map will be discussed in detail subsequently.

The information conveyed by the photographs was largely satisfactory, and more than ample for the purposes of the map. No difficulty was experienced in identifying objects on clear prints, and only a little practice was required in order to interpret the significance of all that showed in the photographs. The most prominent features were the river outlines, the highways being next; railroads were less conspicuous, but were easily recognized by the regularity of their alignment. Cultivated lands, outlines of wooded areas, fence lines, lone trees, and buildings showed plainly. Perhaps the least distinct features were the clay roads and the smaller drainage courses. Among the objects that could not be distinguished and that had to be located by survey were newly built fences and power-transmission lines. The location of power lines could be traced in places by the cleared rights of way through timbered areas. In general, the photographs contained a wealth of detail.

It is to be regretted that the camera was not equipped to record automatically on each photograph the amount and direction of its tilt and the altitude registered by the altimeter. This is now done in the latest equipment and would have been of the greatest value in the orientation work.

The photographic prints used on the Tennessee River survey were of two kinds: A glossy print that showed minute details and was suitable for all purposes except field sketching; and a dull print made on No. 6 P. M. C. bromide paper, which was especially suitable for drawing with ink or pencil, and was used on the plane-table. The bromide prints were not as rich in detail as the glossy prints, and when made from weak negatives gave poor definition. To overcome their limitations, it became customary to provide the men in the field with glossy prints for reference.

Surveying Operations.—The surveying operations are of particular interest, because of their simplicity as compared with standard methods of surveying, such as would have been used normally to produce a map equal to the one obtained. Thus, in the matter of horizontal control, the aerial photograph supplied all that the surveyor required for locating his instrument station.

Triangulation, base lines, control traverses, and the like were unnecessary. The control needed for locating the photographs with respect to each other, under this system, became a drafting-room procedure and will be described subsequently under the head of "The Bagley Method of Orientation."

The vertical control of the Tennessee River work was largely available from previous surveys made by the U. S. Engineer Department, and consisted of permanent bench-marks at intervals of 3 or 4 miles along the river. Check lines were run connecting this old level system (established in 1891) with the precise bench-marks of the U. S. Coast and Geodetic Survey, and except for occasional minor adjustments, it was found to be entirely serviceable. A number of short level circuits were run up tributary valleys, wherever such valleys required mapping for any distance. For the most part, however, height of instrument elevations for use at the plane-table were carried forward by stadia and vertical angles, always commencing at some permanent bench-mark and closing on such a bench-mark. Such vertical angle traverses seldom exceeded 4 miles in length and usually checked within 0.5 ft.

From the foregoing, it will be seen that, barring a small amount of secondary leveling, the man in the field could devote practically all his time and energy to the contour work. The latter was done with the aid of telescopic alidades mounted on standard plane-tables. The process consisted in determining, by the ordinary stadia and vertical angle method, the location and elevation of important points needed for sketching the contours, and platting these points immediately on the aerial photograph. The photograph was mounted on the plane-table as a sheet of paper on which a map is being platted would have been, with this difference, however, that instead of being a blank sheet it showed the location of objects as seen from the air. This was of great assistance to the topographer, and saved him the trouble of locating such objects. Besides the saving of time effected, it facilitated the work in other ways. First, it enabled the topographer to set his plane-table at practically any desired point by identifying it on the photograph with reference to natural objects. Often a cross-roads, a lone tree, an angle in a fence line, or a bald spot in a field, would serve his purpose. Sometimes an unimportant object like a small bush or a rock, showing plainly in the photograph, served as an instrument station. Where no convenient object could be found, stadia distances taken to near-by objects completed the location quickly. The process of orienting the photograph consisted in sighting the alidade at distant points within the scope of the photograph. The usual three-point problem and many of the vexatious troubles of orientation were avoided.

Perhaps the most important advantage of locating contours on an aerial photograph was the assistance derived from being able to see the objects on the ground portrayed in the photograph. This gave the topographer opportunity to check the location of many of the stadia shots as he platted them by reference to bushes, trees, and other objects. In sketching the contours, he had greater certainty of obtaining the correct alignment of minor bends and angularities in such lines than if he had been sketching on blank paper. The assistance derived from the photographs proved especially helpful in eliminating the tendency of beginners to generalize topographic features by conven-

tionalizing the contours. This fault is easily acquired in field sketching and is usually traceable to inability to see the terrain correctly. The foreshortening of the landscape in the eye of a man standing on the ground is a great obstacle to first-class work, and to overcome it, topographers frequently find it necessary to carry the table to elevated points, in order to do the sketching. Probably the most difficult outlines to reproduce accurately on paper are the shore lines of ponds and rivers, which the eye cannot delineate except from highly elevated points, or by making numerous stadia locations. The aerial photograph eliminates this difficulty, for it shows the shore lines correctly, and it was necessary only to determine a few elevations at controlling points. In a number of cases, comparison of the photographic map with the old river charts showed slight errors in the latter, due to misinterpretation of the curvature of bank lines between stadia locations. Such errors are practically unavoidable even in the best of surveying practice. The aerial camera supplements this deficiency.

Another advantage of the use of the photograph is that the elevation of any readily identified object can be obtained without the use of the stadia rod, by merely scaling the distance to it on the photograph and reading the vertical angle with the alidade. This, in many respects, is similar to the intersection method of locating points in ordinary plane-table surveying, except that, in this case, the point is already located and only the computation of its elevation is necessary. This method is not recommended for long-range work, as displacements caused by elevation may introduce appreciable errors in the distances scaled.

Some idea of the relative amounts of information from the aerial photographs and through instrumental survey can be obtained from Fig. 18, which is a reproduction of an aerial photograph exactly as it was received from the field party at the Chattanooga Office. The contour lines on the original are drawn in brick red, the small streams in violet, and all cultural details in black. The part supplied by the surveyors consists of the contour topography and the classification of lands. The latter is indicated by capital letters enclosed in circles and refers to the areas within contours. The notation used is that mentioned on page 782. The data derived from the photograph consists largely of fence lines, water-courses, roads, buildings, and such other important detail as it seemed desirable to embody in the map. It is to be noted that none of this information required any instrumental work by the surveyors. At the most, an occasional reconnaissance was required by one man, in order to make sure of the location of an important fence line through some woods, or to determine the location of a power-transmission line. As the topographic survey was not carried higher than 60 ft. above low water in the river, the area that had to be contoured comprised only 10 to 25% of the finished map. Considering the fact that the control was also derived from the photographs, it will be seen that the field work, although furnishing most valuable data, contributed only a fraction of the information shown; in other words, the photographs alone furnished complete data for a plane map, and all that

required surveying were those parts that were of immediate interest in the planning of dams.

Two additional duties imposed on the topographers are not illustrated in Fig. 18. One of these duties was to make determinations of azimuth on Polaris at intervals of about 10 miles, and the other was to test the scale of each photograph by making stadia check measurements between objects 1 500 to 2 000 ft. apart, that could be clearly identified on the photographs and in the field. These check measurements were of great importance, and in the case of any photograph that was not true to scale they had to be repeated in order to test different parts of the photograph. This was necessary not only for the guidance of the topographer in platting stadia distances correctly, but also for making proper allowances in the construction of the skeleton control by the Bagley method of orientation.

Incidental to the survey was the location and determination of the elevation of reliable high-water marks. To facilitate this, the topographers were equipped with river profiles showing the approximate heights attained by certain great floods, compiled from data obtained in previous river surveys. The field organization consisted of one unit, comprising a chief of party, two plane-table topographers, two station assistants, four rodmen, a launch runner, a cook, and a helper. The entire party was housed in a well equipped houseboat which was moved from 5 to 10 miles at a time, the survey progressing in a down-stream direction. A gasoline launch supplemented by a rowboat was used for taking the surveyors back and forth to work and for transporting supplies. The size of the field unit was determined largely by financial considerations. If the survey should be resumed with ample funds, it is planned to increase the field unit so as to comprise three instrument parties and a field draftsman working under the direction of one chief.

Besides doing the administrative work and providing necessary supplies, it was the duty of the chief of parties to plan the work, to determine what part of the area shown on a photograph was to be contoured; and to make reconnaissances for various purposes, including the determination of elevations of distant sink-holes (with aneroid barometer), in order that no area to be contoured might be overlooked. Finally, it devolved on him to ink the contour lines platted by his topographers and to supply on the photographs other information needed for completing the maps. This included inking in roads, railroads, fence lines, power-transmission lines, buildings, and any other features of importance, as shown in Fig. 18. The putting of the final touches on the photograph in the field was found to be of great utility, for, in this manner, it was possible to go over every contour line in detail, note the discrepancies and omissions, and have the proper corrections made before the photographs were finally transmitted to the Chattanooga Office. With three topographic parties in one unit, it would become necessary to have these duties performed, in part, by a competent draftsman, in order to relieve the chief of parties of too much detail. It may be of interest to state that, in the working plan as first organized, the photographs bearing the contours in pencil were sent to the Chattanooga Office and there inked. The later plan proved the more satisfactory, for the reasons stated.



FIG. 18.—REPRODUCTION OF AERIAL PHOTOGRAPH AS RECEIVED FROM FIELD PARTY, TENNESSEE RIVER SURVEY.

The Stereoscope.—Considerable use was made of the stereoscope on the Tennessee River survey, in both the field and the office. The reflecting-mirror type, without lenses, known as the Pellin stereoscope, was found to be best suited for the work, and was used in two forms, the regular, metal-frame, Pellin instrument, and a home-made box-like structure. The latter required artificial illumination and was used with success in the office. Except for the illumination feature, it possessed decided advantages over the regular Pellin type.

The stereoscope brings out the relief of any area common to two overlapping photographs in a most striking manner by exaggerating the vertical scale. This was found to be especially helpful in country of low or moderate relief, for instance, in tracing minor drainage courses the location of which was often indistinct and difficult to follow. It was useful in examining densely forested mountainous areas which, because of the uniformity of the foliage, afforded no clue regarding topography or drainage when viewed in single photographs. In at least one instance, the presence of deep sink-holes was detected in a hilly and timbered area where the survey party had not suspected the existence of low ground. The discovery having been made in the Chattanooga Office with the aid of a stereoscope, instructions were sent to the field party to make search at points indicated on a photograph. With this information the party had no difficulty in locating the depressions and including them in the survey.

Some of the many practical advantages derived from the use of the stereoscope are as follows: In the field, the topographer was enabled, before beginning a day's work, to visualize clearly the character of the topography of the country. Often it showed him the best places to occupy with his instrument. The chief of party, by studying in the stereoscope the country to be mapped, could plan the work more intelligently. In the office, it was possible, by viewing in the stereoscope the photographs on which contour lines had been drawn, to check the accuracy of the sketching. In many other ways, the stereoscope proved so valuable that it soon was considered an essential part of the equipment.

Office Operations.—The office work was wholly unlike that involved in the preparation of any ordinary map. It consisted in taking a set of duplicate photographs and constructing from them the so-called "skeleton control" which, as the words imply, is a skeleton or frame used for assembling the photographs in proper position with respect to each other. The necessity for providing this skeleton control will be apparent when it is remembered that each photograph contained only a small area, averaging perhaps less than 10 sq. in. of data to be traced. It would have been impracticable to trace a succession of such patches by merely fitting or piecing them together without, in so doing, accumulating serious discrepancies in the resulting map. After testing various methods of building a skeleton control without using data other than those furnished by the photographs, the preference was finally given to the Bagley method of orientation, which will be described later. Once the skeleton control is satisfactorily prepared, the tracing of the photographs be-

comes comparatively simple. One at a time they were placed under the tracing cloth, oriented by the control points provided by the Bagley method, and traced. For photographs that were materially out of scale, it often became necessary to provide additional control points. By centering the tracing cloth over one such point at a time, the data immediately surrounding it were traced, this operation being repeated at the other control points. The intervening detail was then drawn by distributing the discrepancies due to errors in scale, in proportion to the total errors found in the distances between control points. The discrepancies that had to be adjusted by this process were usually quite small and, by providing a sufficient number of control points, it was possible to obviate the use of proportional dividers.

It is believed that the method herein outlined is superior to the use of the pantograph. The lines produced by the pantograph are somewhat ragged, depending on the skill of the operator, and this requires that they be smoothed out by hand, which is objectionable in that it introduces an added source of error in delineation. The pantograph cannot be used to advantage with tilted photographs, as the scale is not the same in different parts of the same photograph.

Experiments were made with celluloid sheets, both with and without ground-glass finish, as substitutes for tracing linen, but the latter was found to be the most satisfactory. The celluloid, although superior in transparent qualities, was difficult to manage, because it would not lie flat and, also, because erasures or perspiring hands ruined the ground-glass finish. The celluloid with glossy finish was not suitable for neat work with India ink. In its final form, the map was made in sheets of standard size, 27 by 40 in. over all, comprising areas of about 6½ by 10 miles each.

Fig. 19 is a reproduction of a part of the completed map, about two-thirds actual size, and is representative of the average map turned out. In addition to the contours along the river and creeks, it shows the location of important roads, fences, buildings, several miles of high-potential transmission line, the position of bench-marks, and distances, in miles, measured along the navigated channel of the river. The classification of lands mentioned on page 782 is not indicated, but will be shown by a colored overprint on the published map. In making studies for water-power projects, this form of map has been found to be convenient, adequate, and accurate. For the use of engineers it possesses inestimable advantages over other types of aerial maps, in that it is made on tracing cloth and, therefore, requires no novel treatment as regards handling, reproducing, or filing. No highly specialized skill is required in its preparation in the drafting room, the only feature requiring some training being the application of the Bagley method of orienting the photographs and correcting their various errors.

The Bagley Method of Orientation.—In 1920, Maj. James W. Bagley, Corps of Engineers, U. S. Army, to whom this country is indebted for much of the progress that has been made in the development of aerial photographic surveying, first tested at McCook Field his new "radial line method" of obtain-

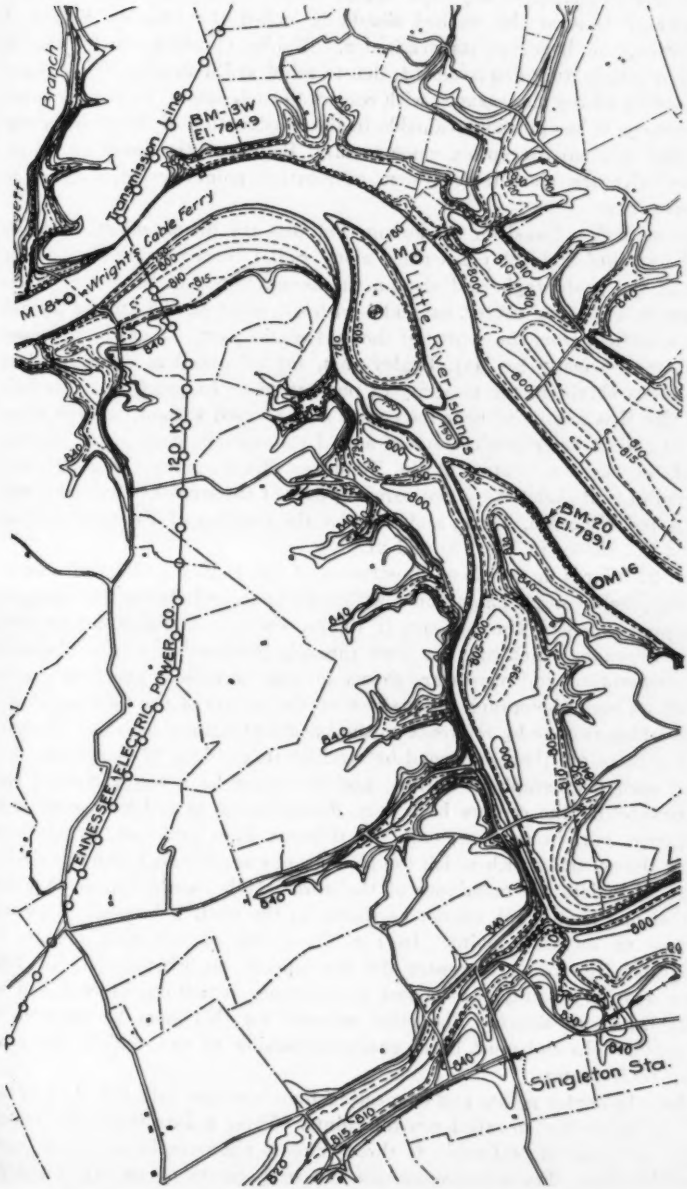


FIG. 19.—SECTION OF COMPLETED MAP OF TENNESSEE RIVER SURVEY REDUCED ABOUT TWO-THIRDS.

ing the correct position of any object shown on two or more aerial photographs. The speaker believes the method should be called the "Bagley Method of Orientation," in honor of its originator. Besides effecting the location of objects by eliminating displacements due to relief and distortion, it facilitates the orienting of the photographs with respect to each other. In the Tennessee River survey, it has proved invaluable in the construction of the skeleton control. The original conception came to Maj. Bagley as the result of his experience with the plane-table system of locating points by intersection, in Alaskan surveys.

The method is based on the assumption that the displacement of objects caused by relief and tilt takes place along radial lines passing through the center of the photograph, and also on the assumption that the center of the photograph, or optical center, coincides with the point on the ground through which a vertical from the center of the lens would pass. It has been demonstrated geometrically by Maj. Bagley that, for all practical purposes, these assumptions obtain in the majority of photographs. In practice, it has been found also that any point near the center may be used without sensible error. Figs. 20 and 21 are reproductions of actual photographs used in the preparation of the skeleton control of the Tennessee River survey. Fig. 22 shows a portion of that skeleton control covering part of the stretch of river between Loudon and Kingston, Tenn., and includes the position of Photographs Nos. 271 and 272 shown in Figs. 20 and 21.

Fig. 22 affords an excellent illustration of the Bagley method of orientation as applied to single photographs within a course, and also to the joining of overlapping courses. The first step in the process is to establish the center of each photograph, which may be done through intersection of the diagonals when no marginal indentures are shown for that purpose. The next step is to locate on each photograph the position of the centers of the adjacent photographs, which is done by reference to the images of natural objects. The successive centers are then connected by straight lines. Fig. 22 shows the position of each photograph in outline, and its center by a heavy circle. The lines connecting the centers have been drawn heavy in order to give them prominence, but, in practice, only the thinnest lines are used, as on them depends the accuracy with which the photographs are oriented, and any deviation would vitiate the remainder of the work. If it should happen that the center of one photograph cannot be shown in the next, it becomes necessary to supply an auxiliary point. In Fig. 22, a case of this kind appears in Photograph No. 275. Its center did not appear on Photograph No. 276, and an auxiliary point, also located by reference to natural objects, had to be established, as shown. The point selected for this must be common to both photographs and must lie as nearly as possible on the straight line connecting their centers.

After the center points and connecting lines are once established, the next step is to select the so-called control points. These points must be situated so as to be common preferably to three or more photographs and in no case to less than two. For instance, the control points in the upper left of Fig. 21 are common to Fig. 20. Some of the control points in Fig. 22 will be seen

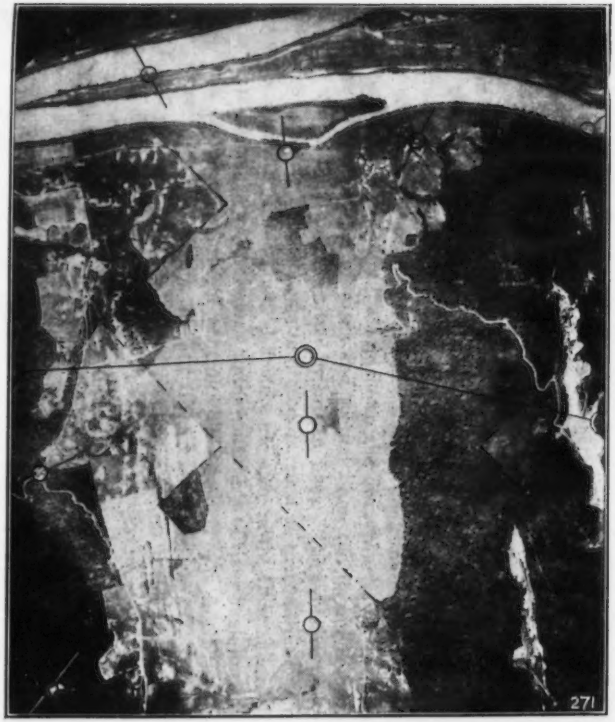


FIG. 20.—REPRODUCTION OF PHOTOGRAPH USED IN PREPARATION OF SKELETON CONTROL, TENNESSEE RIVER SURVEY.



FIG. 21.—REPRODUCTION OF PHOTOGRAPH USED IN PREPARATION OF SKELETON CONTROL, TENNESSEE RIVER SURVEY.

to be common to as many as five photographs. Radial lines are drawn from the center of each photograph through the control points, as illustrated in Figs. 20 and 21. The skeleton control is then constructed by placing the photographs, one at a time, under a sheet of tracing cloth, or celluloid, on which is then drawn the centers and connecting lines, and the radial lines to the various control points. Each successive photograph is placed in position by means of its center already located from the previous photograph, and by orientation on the line connecting its center with that of the previous photograph. As the radial lines to the control points are traced and made to intersect, the proper location of these points is thereby automatically established.

It has been found useful to have from nine to twelve control points to each photograph and to locate them away from the centers, and preferably at or near important features to be embodied in the map, such as shore lines of rivers, highways, and fences. This insures the correct position of these features in the skeleton control. If the photographs are free from tilt and are all taken from practically the same altitude, the radial lines will be found to intersect at single points. It will be noted in Fig. 22 that the intersections obtained were satisfactory in most instances and that points common to both courses showed substantial agreement. In a few cases, the radial lines failed to intersect at a point, and a small triangle was formed. Poor intersections of this kind were usually attributable to distortions caused by tilt. In such cases, the true position of the control point was taken as lying in the center of the small triangle.

In the lower tier of Fig. 22, the radial lines are not drawn so as to intersect. Instead, small circles are drawn indicating the actual positions of the control points in the separate photographs. This was done for the purpose of illustrating the discrepancies that were commonly found in the position of such points. The amount of the discrepancy should be measured between the center of the circle and the point of intersection of the radial lines. Measured by the scale of the photographs, it seldom was as much as 100 ft., and this represented the combined effect of altitude and tilt near the edges of the photographs where errors from these sources would naturally be greatest. In Photograph No. 272, Fig. 22, are shown the discrepancies of this kind as they affected the location of roads and section lines, the former being indicated by double lines and the latter by broken lines. The evidence presented shows the necessity for a systematic procedure, such as that outlined, to eliminate discrepancies of this kind. Efforts were made in the Chattanooga Office to locate photographs with reference to the continuity of river-bank lines, using such lines as the sole form of control. This method was fairly successful when there was no distortion due to tilt, because the surface of the river was, for all practical purposes, a level plane, and points in this plane were not affected by elevation displacements. The Bagley method proved to be superior, however, and was adopted.

Tilted photographs were readily recognized as they would be out of line when platted on the index sheets or in the skeleton control. Thus, in Fig. 22, Photographs Nos. 272, 273, and 275 are probably affected by tilt. Photograph No. 255 is probably similarly affected, but to a lesser degree. In the prepara-

tion of the skeleton control, care had to be exercised to guard against errors in scale. Therefore, before beginning such work it was of much value to be able to note the results of the check measurements made in the field. When a series of photographs of practically uniform scale were found, they were platted at once in skeleton form. If an adjacent course of photographs happened to be on a different scale, that course was platted on a separate sheet, and no matching of the two courses could be undertaken until the proper reduction or enlargement had been made.

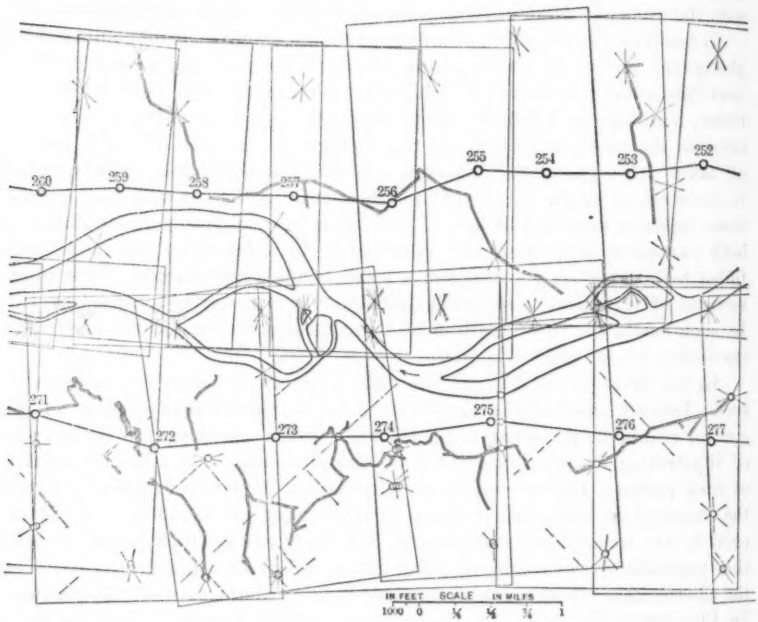


FIG. 22.

In some localities, section lines proved of much assistance in checking the scale of the photographs. It is to be regretted that only a small part of the country was sectionized, and as this had been done prior to 1851, it had become largely obliterated. It is interesting to note, in this connection, the value of the photographs in revealing the remnants of this old system of land lines. In Figs. 20 and 21, the broken ink lines were drawn to mark all plainly visible property lines that were in accord with the sectionizing. Fig. 22 shows, in the lower tier, the character of the information thus made available.

In preparing the skeleton control, it was customary to draw the river banks, highways, and railroads at intervals, because such data assisted in placing accurately the photographs to be traced. Fig. 22 illustrates the extent to which this practice was carried.

It is beyond the scope of this paper to discuss the finer points involved in the use of the Bagley method of control. The experience in Tennessee proved that the method was not difficult to learn, and gave satisfactory results whenever good technique was used and the photographs had ample lap.

Fig. 22 is of interest in showing what to avoid. The two courses of photographs overlap in the region which, in this survey, happens to be of greatest importance, namely, along the river. As the discrepancies due to tilt and elevation displacements are always greatest near the edges, much extra work was entailed in this instance in determining the correct location of the river banks. It would have been better if the aviators had secured the location of the river in the middle of one course of photographs with additional courses on each side.

Ground Control.—The original plans for the survey of the Tennessee River contemplated the use of existing river charts of the Engineer Department as a ground control for the location of the photographs. These river charts were the results of transit surveys made by standard methods, platted to a scale of 1 : 2 000 and reduced by pantograph to 1 : 6 000. Two stadia traverse lines, one along each bank and interconnecting at intervals so as to form a series of closed circuits, furnished the control. With occasional exceptions, these charts were fairly accurate, and afforded reliable checks on the alignment of the photographs. The checks thus obtained demonstrated that wherever ample lap existed between photographs, say, from 50 to 60% in both directions, the map as constructed from aerial photographs by the Bagley method was as accurate as the survey charts, and, in places, the photographic map was the more reliable. The errors that grew out of the skeleton-control method were negligible for the purposes which the map was to serve. The latter consideration was the only criterion by which the sufficiency or insufficiency of the accuracy obtained was judged. Table 5 shows some of the principal check measurements on which the foregoing statements are based.

TABLE 5.

Description of distances.	1909 survey: Scale, 1 : 6 000, in feet.	New map: Scale, 1 : 15 000, in feet.
Out-to-out of north and south loops near Post Oak Island	14 630	14 375
Width of Post Oak Island	2 420	2 450
Length of Post Oak Island	6 500	6 560
Across neck, west of Post Oak Island	3 800	3 750
Across neck, west of Post Oak Island, including island and river	7 520	7 438
Foot of Post Oak Island to foot of Prater Island	18 530	18 562
Across neck at Keller Bend	4 080	4 125
Across widest part of Keller Bend	4 890	4 850
Out-to-out bend at Prater Island to bend west of Cox Island	17 700	17 750
Perpendicular to Louisville Ferry	8 590	8 588
Foot of Prater Island to Point A (near Concord)	20 375	20 375
Across bend south of Concord, Points A to B	7 610	7 460
Across neck south of Concord, out to out	3 880	3 850
Across neck south of Concord, inside to inside	2 450	2 440
Point *O* near Concord, to Point *P*, near Chota Island	19 770	19 900
Point *P* to Louisville Landing	30 730	30 750
Foot of Prater Island to Louisville Landing	11 270	11 250

Maj. Bagley has ascertained that the orientation of a set of photographs possessing at least 50% lap, can be carried in the direction of flight, by his

method, over distances as great as four times the width of the area photographed, with satisfactory results. In the Tennessee River survey, the width was never less than two photographs, or 3 miles wide, allowing for 50% lap, and was usually much wider than that. As a 27 by 40-in. sheet covered at the most about 10 lin. miles of map, the chances of accumulating an appreciable error within the confines of one sheet of standard size were well within the limits stated.

Cost.—The field work on the Tennessee River survey was conducted over a period of 8 months, which included the winter months, and was stopped on June 30, 1922, because of lack of appropriation. In all, 80 lin. miles of the valley of the Tennessee River, measured along the thread of the stream, had been covered. An average cost per square mile for the entire survey cannot be obtained as much of the early work was experimental. Weather conditions, the seasonal state of the vegetation, and the character of the topography introduced wide variations. Toward the latter part of the work, when a regular routine had become established, the indications were that the cost per square mile was about as follows: All field costs, inclusive of field drafting, less than $50; preparation of skeleton control, about $1.25; and final tracing, $3.50. The cost of the aerial work, not including depreciation of equipment and overhead expenses, was about $6 per sq. mile. On this basis, the total cost is about $61 per sq. mile.

Errors in Photographs.—Much apprehension has been expressed concerning distortion and displacement in aerial photographs. The speaker's experience leads him to the belief that such apprehension is frequently the outcome of lack of familiarity with the practical utilization of aerial photographs, and that the net result of such errors is likely to be exaggerated. In aerial photography, as in ordinary ground-surveying operations, instrumental errors are of several kinds and must constantly be guarded against. However, the aerial method being largely a mechanical procedure, is freer from the vexatious human blunders so common in ordinary surveying, blunders which are not easily eliminated, and which in the best accepted practice have to be disposed of by so-called adjustments. The photographic errors in the work done on the Tennessee River were either readily detected and allowance made therefor, or were so small as to be of no importance to the purpose of the final map. The troubles that required the most vigilance were:

1.—Differences in scale of photographs due to the aviators not flying at the proper altitude. Errors of as much as 1% in scale were frequent, but usually such errors were detected readily by comparison with overlapping photographs and through the check measurements made by the field men, as previously described, and also occasionally by reference to section lines.

2.—Displacement of objects due to differences of elevation of the terrain is most troublesome near the edges of a photograph. With this in mind, the areas selected for contouring were made to occupy, as nearly as practicable, the center of the photographs. The outlines of the Tennessee River were found to be quite free from displacement, because its water surface was practically a horizontal plane. This was the more fortunate as the correct loca-

tion of the river not only was of prime importance, but also made possible a close comparison with the earlier ground surveys. The displacement of objects away from the river on ground 200 to 300 ft. above the river level rarely amounted to as much as 100 ft. laterally, and, ordinarily, was much less. As such displacement was always in a radial direction away from the center of a photograph, usually little doubt existed as to its nature and effect. The radial method of locating control points, described previously, proved satisfactory in eliminating displacements which form of error, although the most common to be dealt with, was the least difficult of correction.

3.—Distortion is a term often misapplied to displacement, but relates to entirely different sources of error. True distortions are caused by the tilting of the camera at the instant of exposure; by sudden gyratory movements of the plane; and by imperfect action of the focal plane shutter. Occasional instances were found of these troubles, which would have been quite difficult to eliminate had it not been that, through the use of overlapping photographs, the distorted photographs could be corrected or discarded.

The amount of tilt in aerial photographs is much less than is commonly supposed. In *Information Circular No. 184* of the Air Service are given the results of tests from which it appears that about 29% of the photographs tested showed a tilt between 0° and 0°30′; 64.5% between 0°30′ and 1°; and 6.5% between 1° and 1°30′. A tilt of 3° or more is of rare occurrence. The errors introduced by tilting within the limits stated, affect the position of objects mainly in a radial direction with reference to the center of the photograph, the angular discrepancy being extremely small.

Overlap.—The experience gained on the Tennessee River survey points to the necessity of securing a generous overlapping of the photographs. A lap of about 60% in the direction of flight and of about 50% between courses, is needed, in order to:

(*a*) Construct a rigidly accurate skeleton control;
(*b*) Correct the displacement of objects;
(*c*) Eliminate distortion;
(*d*) Bring out clearly important details by photographing them from more than one angle of view; and
(*e*) Get the full benefit of the remarkable stereoscopic properties of aerial photographs.

Applicability.—The method of aerial photographic mapping, as described, is adapted to any region of moderate relief, regardless of its elevation above sea level. Deep canyons and precipitous mountains introduce troublesome amounts of displacement, and, therefore, aerial photography is not well suited to the mapping of such features. However, it would be practicable to use it in reservoir surveys in mountainous country for the reason that reservoir sites always occupy valley bottoms, and no large part of the adjacent mountains need be shown. This is also true of irrigation projects, because the lands to be irrigated are usually of low relief even though situated in a mountainous region.

The method possesses many advantages in studying routes for power transmission lines through rough or timbered country by eliminating costly pre-

liminary surveys. In such studies, the bare photographs, unassembled in mosaic or map form, will prove helpful in showing all those features which enter into the consideration of locating such lines, not the least being the property lines and the roads and trails required for construction purposes and for patroling. The stereoscope will solve many questions regarding the topography of any proposed route. No ordinary survey or map will furnish all this information so completely or compactly, nor could any surveyor hope to secure so much information at any reasonable expenditures of cost and time.

A valley or canyon presenting a series of alternative sites for dams is best studied by means of aerial photographs. This will save much of the ground surveying ordinarily required in selecting the best site, and the survey of the latter will be facilitated by having the aerial photographs available for mapping purposes. These are only a few of the many uses of aerial photographic surveying that have suggested themselves. The results obtained on the Tennessee River and elsewhere indicate that the time is not far distant when aerial photographic surveying will displant many of the time-honored methods of surveying and map making now in use. Much is being done to improve aerial and photographic equipment, and great progress may be expected, especially if the modern aeroplane should be superseded by a type of aircraft better suited to photographic work.

Acknowledgments

The survey of the Tennessee River was made under the general direction of Maj. Harold C. Fiske, Corps of Engineers, U. S. Army, with the speaker in immediate charge. The general plan which was followed, and which has been described, was first conceived by Maj. Fiske. It is only fair to state that it was due to his resourcefulness and keen personal interest in the survey and the development of its details, that it was possible to accomplish so much with the small funds available. The speaker also takes pleasure in acknowledging his indebtedness to Maj. Bagley for many valuable directions and suggestions. The Army Air Service deserves commendation for the co-operation extended and for the courteous and accommodating spirit displayed by the officials, as well as for the excellent aerial work done by the aviators. A tribute is also due the field survey personnel for its work.

HYDRO-ELECTRIC DEVELOPMENTS ON THE PACIFIC COAST

By John D. Galloway,* M. Am. Soc. C. E.

Synopsis

This paper deals with the general characteristics of power systems in the Pacific Coast States. A short review of the historical development is given, including the increase in size of generators and the parallel development of impulse wheels and turbines, together with the evolution of transmission lines and line potentials. Consideration is given the elements of design and the limitations of impulse wheels and turbines, the tendency toward larger units and the reasons therefor, and the present stage of development of other parts of the power system. The paper also refers generally to the variation in demand of various power systems, the annual increase in demand, the relation of steam plants to water-power plants, and the special relation of hydro-electric plants to irrigation.

Historical

Before discussing some of the problems that enter into the generation, transmission, and distribution of hydro-electric power on the Pacific Coast, a review of the historical development of the art may be of interest as illustrating its rapid growth and the evolution of the design of its various elements.

The first long-distance transmission of energy by electricity was accomplished in Italy in 1886, when single-phase current was transmitted 17 miles, at 2 000 volts, from the Cerchi-Tivoli steam plant to Rome. In 1892, the plant at Tivoli was enlarged by the addition of four 250-kw., three-phase generators driven by hydraulic turbines, and three-phase current was transmitted at a line pressure of 10 000 volts.

In September, 1891, a three-phase transmission was installed from Lauffen to Frankfort, Germany. This line was 110 miles in length and was operated at 12 000 volts.

The first transmission system in America was that built at Oregon City, Ore., on the Willamette River, in 1889. Energy generated by six turbine-driven, 80-kw., 4 000-volt, single-phase generators was transmitted, at the generator voltage, to Portland, Ore., a distance of 13 miles. This development was followed by the building of several small, single-phase plants in the West, namely, at Ames, Colo., and at Pomona, Calif., in 1890; at San Bernardino, Calif., in 1891; at Walla Walla, Wash., and at Bodie, Calif., in 1893.

The first three-phase plant in America was that at Mill Creek, Calif., where two 250-kw., 2 400-volt, three-phase generators were installed in 1892 and placed in operation on September 7, 1893, delivering power 23 miles to Riverside, Calif., at a line potential of 10 000 volts.

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The superiority of three-phase current over single-phase or two-phase current was soon realized, having been indicated by Tesla, Ferraris, and Bradley as early as 1887. After the Mill Creek plant had been placed in operation, three-phase systems became the standard. Many plants were built in the West during the period, 1895-1900. By the end of 1900, transmission lines had reached a length of 86 miles, from Mill Creek to Los Angeles, Calif. The highest line potential, 40 000 volts, was used on the Provo-Mercur line in Utah.

It is unnecessary for the purposes of this paper to recite the details of the progress, except to note that line potentials were raised, successively, to 60 000 volts on the Colgate-Oakland line and on the Electra-Oakland line, 150 miles, in 1901; to 100 000 volts on the Las Plumas-Oakland line, 155 miles long, in 1908; and to 150 000 volts on the Big Creek-Los Angeles line, 241 miles, in 1913; and two lines are now being built in California to operate at a line potential of 220 000 volts. A more detailed historical paper was contributed by the speaker to the International Engineering Congress at San Francisco, Calif., in 1915.*

In the evolution of hydro-electric power systems, the development of electric generators is a striking example of rapid growth in size and design in response to a demand. The first machines were small—only 100 to 200 kw.—but the advantage of larger units was soon recognized, and such units were built, successively, until in October, 1895, 3 730-kw. generators, projected in 1893, were placed in operation at Niagara Falls. These remained the largest units until in 1904, a 5 000-kw. unit was placed in operation at de Sabla, Calif. In 1905, a 7 500-kw. unit was placed in service at Niagara Falls; in 1907, four 8 500-kw. units were placed in operation at Stanislaus, Calif.; and, in 1908, four 10 000-kw. units were installed at Las Plumas, Calif. The 10 000-kw. size remained, to a certain extent, standard for a few years, but in 1913, two 12 500-kv-a. units were installed at Drum, Calif., and four 17 500-kv-a. units at the two Big Creek plants. The 17 500 kv-a. units were not exceeded in the Pacific Coast States until 1921, when the units of Big Creek Plant No. 8, of 22 500-kw., began operation. At the present time, three 25 000-kw. units are being installed at Big Creek Plant No. 3, of the Southern California Edison Company. The largest units now in operation on the Pacific Coast are the two 35 000-kw. units at Pit River Plant No. 1, of the Pacific Gas and Electric Company, one unit of which was placed in service in September, 1922.

Parallel with the development of generators and their increasing size was that of the prime movers: turbines, and impulse wheels. Both these devices were in service before they were used to drive electric generators, but since the two have been used in conjunction, a rapid improvement in efficiency, design, and construction of prime movers has kept pace with the similar development of electric generators and has made the latter possible.

The original small electric plants serving some small industrial unit, such as a street railroad, a mine, or a factory, have grown, during a quarter of a

* *Transactions*, International Eng. Congress, San Francisco, Calif., 1915, Vol. VII, Paper 146.

century, to large systems serving millions of people and with interconnected lines extending more than 1 000 miles. No limit has been reached either in size of units or length of transmission lines, although a line potential of 220 000 volts is approaching the ultimate practicable pressure of possibly 1 000 000 volts.

Elements of Design

In a previous paper,* the speaker discussed the elements of the design of hydro-electric power plants, and the detail will not be repeated here. Some reference is made, however, to design as related to the general subject of hydro-electric systems.

In the paper referred to, the limitations of impulse wheels and turbines were examined. A gap exists between the two, that has not been closed. This may be stated generally, that impulse wheels, on account of efficiency and design, are limited to a single nozzle on each wheel, and each generating unit is limited to two wheels, one on each end of the shaft. The limiting relation between size of jet, diameter of wheel, and velocity of water, when taken in connection with the other two factors, places a maximum limit to the size of units driven by impulse wheels. The 12 500-kv-a. units at Drum, driven by two single-nozzle, 8 500-h. p., impulse wheels at 360 rev. per min., under an effective head of 1 345 ft., and the 17 500-kv-a. units at Big Creek Plant No. 2, driven by two single-nozzle, 10 000-h. p., impulse wheels at 375 rev. per min., under an effective head of 1 780 ft., represent about the maximum sizes for such units at those heads. Any larger impulse-wheel-driven units would require higher heads, a greater number of wheels on one shaft, or more nozzles on one wheel. The last device, after trials, has practically been discarded. More nozzles to one wheel or more than two wheels to one shaft complicates the governing mechanism, and this has not been solved in a satisfactory manner.

The turbine as a prime mover, however, has the defect that it is not applicable to small or medium sized units (2 000 to 5 000 kw.), at high heads, when restricted by limiting generator speeds. On a low-head unit, less than 200 ft., the turbine is the only prime mover for units of 5 000 kw. and more. On horizontal shaft units, two runners, and on very low-head plants, four runners, have been used. On medium-head plants (200 to 750 ft.) single-runner turbines are available for most units built in the last ten or twelve years. Examples of smaller sizes with single runners are cited in Table 6.

TABLE 6.

Location.	Horse-power of turbine.	Effective head, in feet.	Speed, in revolutions per minute.
Salmon River N. Y.	10 000	245	375
Snoqualmie Plant No. 2, Wash.	12 500	270	360
Nisqually, Wash.	8 000	425	450
Coleman, Calif.	7 000	487	450
Centerville Calif.	9 700	565	400
Carp River, Mich.	4 000	580	720

* *Transactions*, Am. Soc. C. E., Vol. LXXIX (1915), p. 1000.

The best use for the turbine is found in the larger sized units installed in recent years, of which the list in Table 7 gives some examples of low and medium sizes of units with single runners.

TABLE 7.

Location.	Horse-power of turbine.	Effective head, in feet.	Speed, in revolutions per minute.
Keokuk, Iowa	10 000	32	57.7
Canadian Niagara Falls	10 000	130	250
Shawinigan Falls, Que	18 500	145	225
Laurentide Power Co., Que	20 000	76	120
White River, Wash	18 000	440	360
Las Plumas, Calif	18 000	465	400
Big Creek Plant No. 8	31 500	729	428
Big Creek Plant No. 3	35 000	740–800	428
Pit River Plant No. 1	40 000	450	257
Canadian Niagara Falls	60 000	.....	.....

Thus, with the use of larger units, the turbine is being utilized under higher heads. There is no reason why turbines built with cast-steel scroll cases should not be used under heads up to 1 000 ft., and possibly higher. For higher heads, the impulse wheel remains the only prime mover, and, therefore, where a power station might have large size units, if turbines could be utilized, the use of impulse wheels would materially reduce the size. No limit in size has been reached for turbine-driven units. A single unit of 70 000 h. p. is under construction for Niagara Falls.

The preceding discussion is given to illustrate by examples the limitations of turbines and impulse wheels and also the trend of present practice. As units have grown larger, the turbine has usurped the domain of the impulse wheel for heads up to 750 ft. The growth in size of units has coincided with the increase of power systems. The principal reasons for this result may be stated, as follows:

(*a*) Increased efficiency of the unit.
(*b*) Relatively less first and operating cost.
(*c*) Less cost of power station building.
(*d*) Increased length and potential of transmission lines requiring larger charging current capacity in single-generator units.
(*e*) The increased load on power systems, with greater total generating capacity at several interconnected stations, allowing larger units to be used at any one station.

At present, the largest single units in the West are the two 35 000-kv-a. generators operated by single-runner turbines at Pit River Plant No. 1, of the Pacific Gas and Electric Company in California. Reference may also be made to the use of vertical shaft units that now represent a type possessing many advantages over those with horizontal shafts. The vertical shaft units have been made possible by the use of the Kingsbury bearing, or other bearings depending on the same principle. The entire rotating element, consisting of

the turbine runner, the shaft, and the generator rotor, are suspended on the bearing which is sustained on the top of the generator frame by a spider. This bearing has eliminated the use of the older solid disc bearing, or the roller bearing, both of which required oil under heavy pressure, with the resulting expensive system of pumps.

The principal advantages of the vertical unit over the horizontal may be stated, as follows:

(1) A straight draft-tube is generally possible, giving better discharge and higher efficiency.
(2) The turbine runner is more accessible.
(3) A thrust-bearing is not required.
(4) The generator is in a better position for ventilation and uniform atmospheric conditions and is also more accessible for repairs. The rotor can be lifted from the unit without disturbance of other parts.
(5) The generator can be placed on an upper floor above high water.

Of the later developments in the prime movers may be mentioned the governor-operated, synchronous by-pass on impulse wheels, that has replaced the older deflecting nozzle, and, on turbines, the governor-controlled, direct-connected servo-motors for operating the turbine gates. Direct-connected, synchronous by-passes have been developed in the last fifteen years. These by-passes obviate, to a great extent, the danger to the penstock and turbine from surges due to interruption of the current. A recent device places the rotating element of the governor on the turbine shaft, thus eliminating belt connection. On many later units, the exciter units are direct connected to the generator, being placed on an extension of the shaft, replacing the older exciter units with their water and electric-motor drives. This idea is not new, but it was possible to use it as the power systems became larger.

Coincident with the development of larger turbines has been that of butterfly or pivot-valves that are practically water-tight. Valves of the balanced piston or Johnson type, with the pivot-valves, have replaced the older gate-valves that became impracticable in large sizes under high heads.

Of the other accessories of power plants, the developments of the last five years have brought little change. American-made welded-steel pipe for penstocks in large sizes is a more reliable article than that formerly imported from Germany. Conduits are made more permanent, and concrete-lined tunnels are replacing the open ditches and timber flumes of the past. The theory of surges in pressure conduits and the design of surge-chambers and their installation at the junction of conduit and penstock has become a regular feature of power plants on the Pacific Coast.

At the power plant, all transformers, high-tension switches, and bus-bars were placed at first inside the building. During the last few years, however, there has been an increasing tendency to place this apparatus outside the building, especially where line potentials of more than 100 000 volts have been used. This practice, of which the Southern Sierras Power Company was one of the pioneers, is rapidly extending. On the new 220 000-volt lines of the

TABLE 8.—Daily Average Energy Output, in Kilowatt-Hours, and Percentage of Annual Mean.

Month.	Washington Water Power Company.		Puget Sound Power and Light Company.		Pacific Gas and Electric Company.		San Joaquin Light and Power Company.		Southern California Edison Company.	
	Daily average, in kilowatt-hours.	Percentage.	Daily average, in kilowatt-hours.	Percentage.	Daily average, in kilowatt-hours.	Percentage.	Daily average, in kilowatt-hours.	Percentage.	Daily average, in kilowatt-hours.	Percentage.
January	984 430	96.0	1 196 330	104.1	3 964 000	97.2	778 960	71.8	2 297 360	78.5
February	969 590	94.5	1 177 020	102.4	3 857 000	94.5	765 100	70.5	2 052 010	70.0
March	977 190	95.3	1 182 710	103.1	3 806 000	93.4	821 800	76.8	2 464 890	84.3
April	925 190	90.3	1 137 340	99.0	3 937 000	96.5	1 213 120	112.0	3 085 780	105.8
May	879 980	85.8	1 041 040	90.8	4 290 000	105.0	1 172 890	108.1	2 865 560	98.0
June	973 700	94.9	1 064 120	92.7	4 430 000	108.5	1 258 130	116.0	3 309 960	113.0
July	1 011 440	98.7	1 007 350	87.7	4 284 000	104.9	1 369 600	126.0	3 575 340	122.0
August	1 112 690	108.6	1 078 000	93.9	4 277 000	104.8	1 360 160	125 2	3 548 890	121.0
September	1 076 040	104.9	1 159 530	101.0	4 133 000	101.2	1 193 580	110.0	3 428 730	117.0
October	1 127 680	109.9	1 186 970	103.2	4 048 000	99.2	1 035 700	95.5	2 855 020	97.5
November	1 155 080	112.7	1 274 420	111.0	3 973 000	97.4	1 051 680	97.0	2 911 240	99.5
December	1 110 430	108.2	1 277 890	111.2	3 974 000	97.4	984 940	90.8	2 744 340	93.8
Mean	1 025 690	100.0	1 148 490	100.0	4 081 000	100.0	1 085 410	100.0	2 932 630	100.0
Annual load factor	61%		48.8%		63.6%		52.9%		64%	

Pacific Gas and Electric Company, all high-tension apparatus is placed in the open.

Transmission lines have been increased in length and in line potential. Probably the longest transmission is that of the Southern Sierras Power Company to Yuma, Ariz., more than 600 miles. Steel towers and suspension insulators have become standard. The long lines, high potentials, and large amounts of energy transmitted now require large installations of condensers at the receiving end, an important element in the cost.

Characteristics of Western Power Systems

At the beginning, little was known of the various factors that enter into the generation, transmission, and distribution of hydro-electric power. As the systems extended and a wider range of industries were supplied with power, the characteristics of such industries became better known. The integrated result of load variation, expressed as an annual load factor, or ratio between the yearly average and maximum or peak load, measures, in one way, the demands on a power system. The annual load factors of several large Pacific Coast systems are given in Table 8.

Another important element in the systems, however, is the seasonal variation in energy output or demand, which differs widely on different systems, especially in California. The rainless period, extending over about seven months, causes the Sierra Nevada streams to diminish in summer to a small proportion of the average flow. Again, many industries, such as those that depend on agriculture, are subject to wide variation in power demand (from irrigation pumping, fruit-packing establishments, and coincident activity), occurring at a time of year when the streams are low, and thus require that the power deficiency in natural stream flow be made up by stored water or by steam-generated energy.

Table 8 gives the total energy generated by months, but expressed as a daily average for the month, for two companies in Washington and three in California. Purchased energy is included, as it all represents a demand on the company. The year 1921 was chosen as representing the latest information, and also because, on most of the systems, there was not much increase in demand. A column of percentage departures from the mean is added to express the variation in demand. Fig. 23 shows the percentage variation graphically.

There is an error in Table 8 and Fig. 23, as the kilowatt-hour output necessarily contains the increase in demand that comes to all growing systems. This, however, is a better representation of conditions that will prevail for a considerable time. Attention is called to the range of power demand. Central California, as represented by the San Joaquin Light and Power Company, in the San Joaquin Valley, a community predominantly agricultural, and Southern California, represented by the Southern California Edison Company, have a greater annual variation in energy output than the companies of Northern California or Washington. The examples are given as representing Pacific Coast conditions and as what may be anticipated elsewhere under

similar conditions. It should be noted that the system peak does not necessarily occur in the month of maximum energy demand. The systems discussed serve a great variety of industries, of which mining and agriculture comprise a fairly large part. Lighting requires a relatively small part of the energy sold.

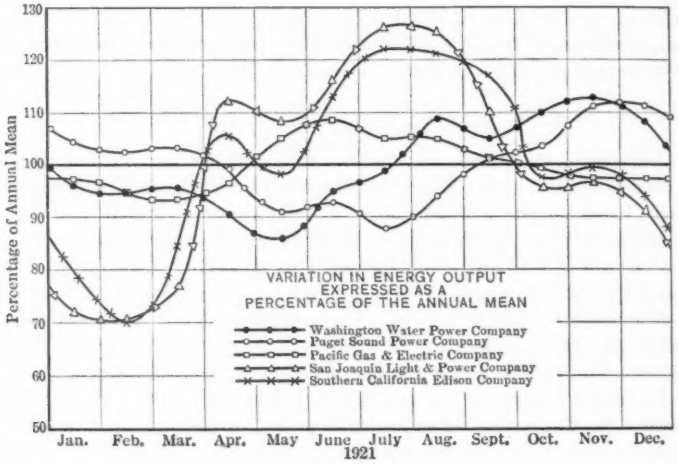


FIG. 23.

Increase in Demand.—The development of hydro-electric power during the last ten years has been on a firmly established basis. It is possible to take the records of growth for that period and to determine with reasonable certainty a rate of growth that will enable one to forecast the future demand for ten or fifteen years to come. Different localities exhibit strikingly different rates. Again, the rate of increase in demand has not been uniform over the last ten years, due largely to the World War and the resulting industrial changes. The average rate, expressed as an annual compounding rate, over the entire period will give a fairly good index of future growth. Table 9 is given to show conditions in California, where the rate for the State is 11.1% and shows an increase in nine years, as follows:

Water.—From 1 068 000 000 to 2 493 000 000 kw-hr., or, approximately, 134% which is 9.9% compounded annually.

Steam.—From 334 000 000 to 1 124 600 000 kw-hr., or, approximately, 237% which is 14.4% compounded annually.

Total.—From 1 402 000 000 to 3 617 600 000 kw-hr., or, approximately, 158% which is 11.1% compounded annually.

It also shows that the steam output ranged from a minimum of 15.2% to a maximum of 36.2% of the total output.

An examination of Table 9 which was prepared by A. H. Markwart, M. Am. Soc. C. E., shows a wide variation in power demand from year to year. Similar tables show for Northern California an annual compounding rate of

6.96%; for the Pacific Gas and Electric Company, a rate of 7.44%; for the Great Western Power Company, 7.07%; and a rate of about 18% for Southern California.

TABLE 9.—KILOWATT-HOUR OUTPUT FROM WATER AND STEAM, 1911 TO 1920, INCLUSIVE, FOR STATE OF CALIFORNIA, BASED ON RAILROAD COMMISSION RECORDS FOR ALL PUBLIC UTILITIES, INCLUDING CITY OF LOS ANGELES AND OTHER SMALLER MUNICIPAL PLANTS.

Year.	Water.		Steam.		Total.		Steam, in percentage of total.
	Millions of kilowatt-hours.	Percentage of increase.	Millions of kilowatt-hours.	Percentage of increase.	Millions of kilowatt-hours.	Percentage of increase.	
1911.....	1 068.0	.....	334.0	.....	1 402.0	....	23.8
1912.....	1 143.0	7.0	445.0	36.2	1 598.0	14.0	28.5
1913.....	1 263.9	10.6	715.2	57.2	1 979.1	23.8	36.2
1914.....	1 746.0	38.1	356.3	50.2 (Dec.)	2 102.3	6.2	16.9
1915.....	1 816.8	4.1	394.5	10.7	2 211.3	5.2	17.8
1916.....	1 992.2	9.6	357.1	9.5 (Dec.)	2 349.3	6.2	15.2
1917.....	2 142.5	7.5	495.5	38.8	2 638.0	12.3	18.8
1918.....	2 254.3	5.2	710.7	43.4	2 965.0	12.4	24.0
1919.....	2 241.3	0.6 (Dec.)	993.6	39.8	3 234.9	9.1	30.7
1920.....	2 493.0	11.2	1 124.6	13.2	3 617.6	11.8	31.1
Total increase....	1 425.0	134.0	790.6	237.0	2 215.6	158.0	....
Annual increase....	.....	9.9	.....	14.4	.....	11.1	....

It might be questioned as to whether this rate of growth is temporary only and whether the market will not be soon saturated. The only way in which this could be ascertained was to examine the rate of increase of population in the United States and to compare it with the growth of standard industries, all expressed as a compounding annual rate. These statistics are given in Table 10.

TABLE 10.—INCREASE OF CERTAIN ITEMS IN THE UNITED STATES.*

Item.	Period.	Compound annual percentage rate.
Population of United States	1900-10	1.95
" " " "	1910-20	1.38
Coal produced	1900-09	8.78
" " "	1909-19	5.71
Pig iron produced	1901-09	6.15
" " "	1909-19	4.18
Steel and iron produced	1900-10	8.60
" " "	1910-20	4.10
All United States manufacturing	1904-14	5.06
Post Office revenues	1900-10	8.16
" " "	1910-20	6.90
Bank clearings, Pacific Division	1911-19	13.40
California manufacturing	1904-14	6.86

* From U. S. Statistical Abstract.

It will be noted that even in the old established industries the annual compound rate of increase is from three to four times the similar rate of increase of population.

The deductions from the study are that, for a long time to come, the power demand on the Pacific Coast may be expected to increase at an annual compounding rate of from 5 to 7%, and much more in some localities. It is probable that by 1930, the annual energy output in California, will exceed 10 000 000 000 kw-hr.

Relation of Steam to Hydro-Electric Power

A proper appreciation of the relation of the steam plant to the water-power plant has arisen with the development of power systems. It might seem at first that when a sufficient amount of generating capacity in the water-power plant had been built, there would be no need of a steam plant. Circumstances, however, have determined otherwise. Everywhere on the Pacific Coast, the hydro-electric plants are situated in the mountains, more or less remote from the cities. Possibly the only exception is at Spokane, Wash., where the Falls of the Spokane River within the city makes it unnecessary for the Washington Water Company to carry any of its load on steam. The opposite extreme is found in California, where transmission lines are 150 to 250 miles long from power plants to load centers. Steam plants, located in large cities, have been built either as safety reserves or to furnish part of the energy.

The functions of the steam plant, acting in its proper relation to the hydro-electric plant, may be segregated under five general divisions:

(*a*).—As a reserve in case of interruptions to the hydro-electric system. In some cases, as in San Francisco, due partly to the situation, the steam reserve carried by the Pacific Gas and Electric Company exceeds 75% of the load.

(*b*).—To generate a substantial portion of the energy output. Fuel costs are such, however, that, with the average installation prices of hydro-electric systems, energy can be developed cheaper by water than by fuel. In California, however, the abundance of cheap fuel oil in the past, ranging in price from 50 to 60 cents per bbl. at the plant, equivalent to good coal at $2.50 per ton, resulted in the building of numerous large steam plants. The capital charges per kilowatt were considerably less and the energy costs were also less. The time of low priced oil passed, and power companies found their energy costs suddenly raised. This has resulted in an extensive program of hydro-electric development. With oil at $1 per bbl. or more, steam-generated energy is too costly.

(*c*).—To generate energy to carry some of the system load over a period when all the water-power plants are loaded and before the demand increases to a point where the large initial cost of a modern water-power plant is justified.

(*d*).—To generate energy in years of extreme low water. Reservoirs, as a rule, can be built to equalize stream flow during most years, but, at times, a season occurs when the stream flow is much below normal. In such cases, the steam plant makes up the deficiency.

(*e*).—To carry a fair proportion of the peak load. In one respect, this is the most important function of the steam plant. The longer the transmission line, the more important does this function become. The maximum peak load

of a year occurs only once and may last for only ½ hour. Other peak loads of other days or months are not as great. If dependence is placed on the water-power plant alone, all equipment, including long transmission lines, must be installed to carry a power peak, the energy content of which is negligible.

In this case, the steam plant can be arranged to carry from 20 to 25% of the peak, without generating a great amount of energy. This arrangement is beneficial in many ways. Besides saving capital costs in regulating reservoirs, penstocks, hydro-electric generating units, and transmission lines, it raises the load factor on such plants and at the steam plants places the units in service each day. In many cases, the power factor of the system is raised by the operation of the steam plant generators.

The characteristics of each power system and the price of fuel will determine the amount of peak load to be carried and the energy generated at each type of plant. The peak to be carried by steam will range from 10 to 25% of the total. The energy output will depend on the shape of the peaks and the relation of the maximum peak to other average peaks, and may range from 2 to 10% of the annual energy output of the system. Thus, on the average system in the West, the daily load factor on the water plants can be raised from about 60% to from 70 to 85 per cent. The daily load factor on the steam plants will be low, ranging from 25 to 35 per cent.

A diagram, Fig. 24, of a daily load curve, divided between steam and water plants, is given to illustrate this subject.

The Relation of Hydro-Electric Plants to Irrigation

Irrigation is practiced in many parts of the Pacific Coast and produces in hydro-electric systems a large part of the excess power demand of summer. In many cases, especially in California, the same stream that furnishes power, is used for irrigation after it reaches the valley lands. Practically, all the important rivers that descend each slope of the Sierra Nevada are thus used.

There is an apparent conflict between the two uses of water. Power demands a fairly uniform flow throughout the year. Irrigation demand varies between wide limits, ranging from 1 or 2% in some months to as much as 20 or 22% of the total seasonal quantity used. During two or three months of the year, there is no irrigation, and then reservoir gates are closed to store water for the next year.

The solution of this problem is found in the difference in reservoir location. Power requires that water be stored near the head-waters of rivers, in order to take advantage of as much head as possible. For irrigation, however, the reservoirs are needed near the exit of the river from the mountains, in order to obtain the greatest tributary water-shed. Whatever water is released from power storage reservoirs during the irrigating season is of benefit to irrigation, passing directly to the lands. Water released for power during the non-irrigating season can be retained in the irrigation reservoir for use the next season, and thus insures against a failure of irrigation water for a possible future deficient flow of the stream. The irrigation reservoir becomes, in effect, a re-regulating reservoir, changing the stream from the character given it by power regulation to one serving irrigation.

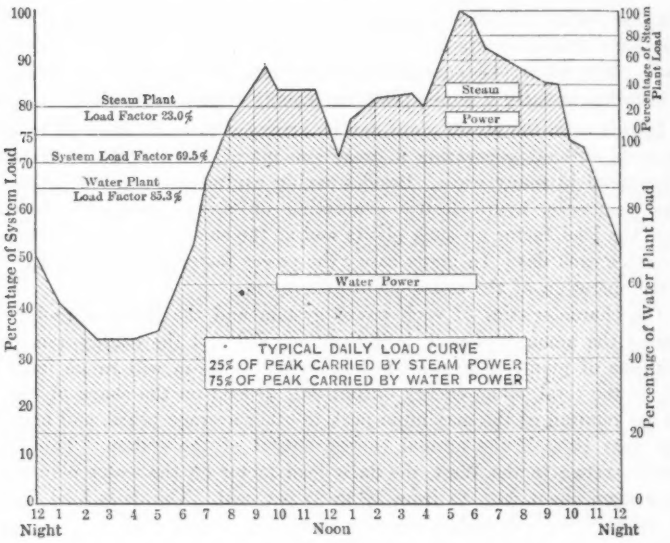


FIG. 24.

These general relations are illustrated in Fig. 25, which shows a hydrograph of a typical California stream, a power demand curve, and an irrigation demand curve, based on conditions in Central California. No fixed rule can be given for solving such problems. Each case must be solved by itself, taking account of the physical characteristics of the stream, the reservoirs, the amount and variation in demand for power, and the amount of irrigation, with its characteristics of use of water, crops, etc.

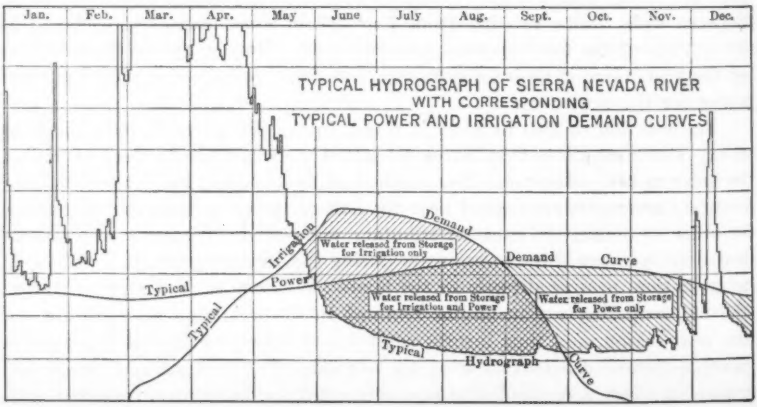


FIG. 25.

A secondary relation between power and irrigation occurs in hydro-electric plants built as adjuncts of irrigation reservoirs. High dams are being built as irrigation storage reservoirs, and this makes possible the development of some power. As the water is used subject to the irrigation demand, the energy output varies widely. At times during the peak of the irrigating season, a very large amount of power is available, but as the duration of this output is short, possibly one or two months, there can be no sale for the maximum amount. Possibly one-third would represent a fair proportion of the maximum possible power to be developed. After the irrigation peak has passed, the power falls off rapidly, owing to decreased irrigation demand and to decreasing head in the reservoir.

These special characteristics make the sale of such power a difficult problem. If used by the irrigation district, that organization must build steam plants to carry the load during the non-irrigating season. If the power is sold to the nearest power system, that system must be capable of absorbing the block of summer power or otherwise it cannot purchase the power. Each case must be considered by itself.

A case in point occurred in connection with the Merced Irrigation District in Central California. In planning the works of that district of 186 000 acres, the speaker planned a reservoir with a capacity of 250 000 acre-ft. on the Merced River. The dam, about 300 ft. high, made possible a power plant of 30 000 kw. and the sale of the energy to the San Joaquin Light and Power Company. The load curve of that Company permitted the absorption of this special block of energy, because the pumping of water for irrigation is a large part of its summer load, and this coincided in time with the delivery of power from the plant of the Irrigation District. Thus, a net revenue was made possible for the Irrigation District sufficient to pay the fixed charges on the cost of reservoir and power plant, amounting to more than $7 000 000.

In the development of hydro-electric power systems of the Pacific Coast States many changes have been brought about. These changes are the joint product of the civil, mechanical, and electrical engineers. As a rule, most of the expenditure comes under the control of the civil engineer, but the engineer in charge of any system must have a fair knowledge of all the problems involved. The speaker must express a high degree of appreciation of the mechanical engineers who have perfected the modern American impulse wheels and turbines and of the electrical engineers who have designed and built the high efficiency and mechanically perfect generators, transformers, switches, and other parts that are a part of the complicated power plant.

HIGH-VOLTAGE PHENOMENA ENCOUNTERED IN POWER TRANSMISSION

By Harris J. Ryan,* Esq.

Electrical power transmission fundamentally involves the care of the following:

1.—A proper cross-section of conductor to accommodate the requisite current.

2.—An adequate non-conducting support and submerging medium for the transmission conductors, that will withstand the application of the necessary high voltage without appreciable leakage of current, that is, an adequate system of insulation.

3.—The stability of voltage values.

The provision of a conductor of proper cross-section is an economic consideration and gives rise to few practical or technical difficulties. Under the usual schedule of costs, the most economic section of copper conductor per ampere, is never much more than that of a No. 18 B. & S. gauge wire. Such a conductor weighs 26 lb. per mile and has a resistance of about 35 ohms per mile. It follows, therefore, that the weight of the copper conductor is about 26 lb. per ampere per mile, and the corresponding voltage required to supply the power lost in such a conductor is 35. This value of 35 volts that must be lost economically in a copper transmission line, is fundamental. Electrical power conductor-transmission efficiency, therefore, can never vary greatly from

$$\varepsilon = \frac{35\ D}{E_0}$$

wherein E_0 is the voltage to ground or to "neutral", used for transmission, and D is the distance, in miles.

The total economic cost of power transmission due to the conductor alone is the sum of the existence cost attached to the conductor section and the cost of the conductor electrical loss. The fundamental relation of transmission voltage, distance, and conductor-transmission cost, is: Total conductor transmission costs vary directly as the transmission distance and inversely as the voltage.

Whenever practicable, the relation of transmission-line voltage to transmission distance is about 1 000 volts per mile in a three-phase circuit (575 volts from conductor to neutral). This results in a power loss of $100 \times \frac{35}{575}$, or 6.1%, and a total conductor-transmission cost, therefore, of 12.2 per cent. Under this allowance of working voltage, water power has been developed during recent years in intermediate amounts for single projects of 100 000 to 200 000 kw. and transmitted to profitable markets over correspondingly intermediate distances of 100 to 250 miles, using voltages from 66 000 to 165 000.

The economic conditions on the Pacific Coast and in other sections of the United States now demand the development and use of all outstanding water

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powers and the transmission of power to profitable markets. Individual projects involve the development of blocks of power varying from 250 000 to 1 000 000 kw., or more, to be transmitted for the greater part, over distances from 250 to 500 miles or more.

When active preparations for the first of these large modern projects were begun in 1919, it was realized that voltages higher than those in use at that time would have to be employed to insure economic and practical success. The technical difficulties involved would have to be solved, and the whole subject of power transmission would have to be studied with respect to the available technical expediencies and practical options of every kind, in order that the next advance in voltage might be made advisedly for the general mobilization of all economic sources of power. During the past three years, much work has been done by power-transmission engineers co-operating for the interests they represented, and by committees of the National Electric Light Association and the American Institute of Electrical Engineers. The result of this effort has been the virtual adoption by common consent of 220 000 volts and the development of dependable line insulators, transformers, and switch-gear required for the application of such voltage.

Many arguments for and against the adoption of 220 000 volts have been considered. The majority of these arguments, although often the cause of much controversy, are of little importance. There are three inter-related factors, or groups of factors, to which the final decision is largely due, as follows:

I.—Economy in transmission, as such, demands the highest available working voltage.

II.—High voltages are extravagant of space, demanding liberal air-space clearances adjacent to all insulator supports of 18 000 volts per ft., for example, 7 ft. at a minimum for the use of 127 000 volts to ground occurring in a 220 000-volt transmission.

III.—For the transmission of a given amount of power, the idle re-active power required to maintain the electric field attached to the transmission circuit is positive and increases directly as the square of the transmission voltage; the corresponding magnetic field is negative and increases inversely as the square of the transmission voltage.

The nature of Factor I will be understood when it is recalled that power costing 0.2 cent per kw-hr. at the large power plant will cost approximately an equal amount to transmit it 300 miles, using 220 000 volts, to a profitable market. The corresponding conductor part of this cost is $\frac{2 \times 100\ (35 \times 300)}{127\ 000}$ = 19.3 per cent. Elevating the voltage, therefore, can effect a reduction only in a one-fifth part of the transmission cost and must simultaneously encounter increased costs on account of Factor II and greatly increased costs on account of Factor III.

Factor II is the effect on the cost of transmission, due to a necessary linear increase of clearances in air, oil, and solid insulations, with a corresponding increase of line voltage. Tower structures, transformers, and switch-gear are

to be had, therefore, at fair unit costs only when the magnitude of power transmitted is sufficiently great per circuit (100 000 to 200 000 kw.), otherwise the saving in conductor-transmission cost is rendered of no value by the accompanying increased cost of auxiliary equipments.

Factor III affects plant capacity (that is, the power factor) and voltage control. When capacitance and inductance are connected in series in the electric circuit, they consume correspondingly positive and negative re-active powers by the production of opposing voltages that are in lagging or leading quadrature with the electric current. If the re-active powers thus consumed are equal, the corresponding voltages produced are equal and opposite. Their resultant is zero. The conductor-transmission power factor remains unity. When such inductances and capacitances are connected in parallel with the electric circuit, opposing currents instead of voltages are set up. If the re-active powers thus consumed are equal, the sum of their corresponding currents is zero. The net result is substantially the same as in the preceding case. If, however, inductance is connected in series with the circuit and the capacitance in parallel, as is the case in the long transmission circuit, the inductive or magnetic and the capacitive or electric re-active powers are consumed by setting up corresponding voltage and current. They are unlike components of re-active power and cannot be added directly without the one being first transformed to the form of the other.

The most practical transformer for this purpose is the so-called synchronous condenser. It should more properly be called the re-active alternator as it may be made at will to accept quadrature leading or lagging current from the transmission line at any point by a proper adjustment of field excitation. In the discussion that follows, it is assumed that re-active alternators of suitable negative kilovolt-ampere capacity are connected across the transmission line at intervals of about 150 miles, as requisite facility for mixing en route through the transmission line the opposing electric and magnetic re-active powers consumed by the line, so as to have to transmit in the aggregate only the resulting re-active power residual and to maintain a constant line voltage and power factor, as proposed recently by F. G. Baum, M. Am. Soc. C. E.*

A 100-mile, 110 000-volt, three-phase, 60-cycle, 32 200-kw., power-transmission circuit requires equal and opposite re-active powers (6 700 kv-a. each) to maintain the electric and magnetic fields attached to such a circuit. This circuit, therefore, will have a conductor-transmission power factor of unity. For these power-transmission values, Factor III is the cause of no difficulty. Unfortunately, when the power is transmitted in a fixed amount, the re-active power required for the electric field increases directly as the square of the transmission voltage and, for the magnetic field, inversely as the square of such voltage, and each increases directly as the transmission distance. Thus, on increasing the voltage from 110 000 in the case cited, the excess of re-active power required for the maintenance of the electric field over that in opposition required for the corresponding magnetic field, increases with great rapidity, causing a corresponding reduction in power factor and voltage

* "Voltage Regulation and Insulation for Large Power, Long-Distance Transmission Systems", *Transactions*, A. I. E. E., Vol. XL (1921), p. 1017.

stability. Only by increasing the power transmitted per circuit can this excess of electric re-active power be limited satisfactorily.

Taking the values in the case cited as units for a comparison of the electric and magnetic re-active powers required for other values of line voltage, E, the line voltage, will occur in 110 000-volt units, the distance, D, in 100-mile units, and the power, P, in 32 200-kw. units, the values of the opposing re-active powers required to maintain the fields attached to the transmission circuit, are:

$$\text{Electric re-active power} = (+)\ \overline{KVA}_e = 6\,700 \cdot D\,\overline{E}^2 \quad \ldots\ldots (1)$$

$$\text{Magnetic re-active power} = (-)\ \overline{KVA}_m = 6\,700 \cdot \frac{D\,\overline{P}^2}{\overline{E}^2} \quad \ldots\ldots (2)$$

By equating these values of the electric and magnetic re-active powers, the corresponding relation of the values of power and voltage in the units adopted is found to be:

$$P = E^2 \quad \ldots\ldots (3)$$

It follows that the power per circuit must increase as the square of the voltage, in order to transmit at a full-load, conductor-power factor of unity. Three of these corresponding voltages and powers are, as follows:

Three-phase line voltage, in kilovolts.	Power transmitted per three-phase circuit, required for a conductor-power factor of unity, in kilowatts.
220	128 000
330	298 000
440	512 000

It is seen, therefore, that as the voltage is raised above 220 000, the magnitude of power per circuit required for a conductor-transmission power factor of unity quickly becomes excessive. It follows, therefore, that 60-cycle, alternating, transmission voltages cannot be used at values of much more than 220 000 without a serious loss of the power factor and a corresponding increase of the non-productive capacity of the synchronous machinery required to maintain the high electric fields that inevitably attach to circuits at high voltage.

The curves in Fig. 26 have been determined to show the approximate relations between magnitudes of 60-cycle power transmitted per circuit, residual re-active powers, and line voltages, for distances of 250 and 500 miles.

If the frequency was lowered from 60 to 25 cycles, each of the opposing re-active powers would be reduced to about 40% of their former values, permitting a corresponding increase in transmission voltage from 220 000 to 350 000. The saving in conductor economy thus effected would be offset so largely by increased costs of transformers, switches, and the greater clearances required for insulation, that the gain would be uncertain. By common consent, therefore, no reduction in frequency has been proposed for the large power projects and for the nation-wide power network.

The curves, Fig. 26, show likewise that a transmission voltage of 220 000 would be unduly high for the satisfactory transmission of power magnitudes much less than 100 000 kw. per circuit, again, because of the relatively exces-

sive values of re-active power required to charge the electric fields surrounding the conductors.

The conclusion is reached, therefore, that the best transmission voltage for the establishment of the general power network is 220 000 at 60 cycles. With such a voltage, power can be transmitted with practical satisfaction over distances from 200 to 500 miles, or more.

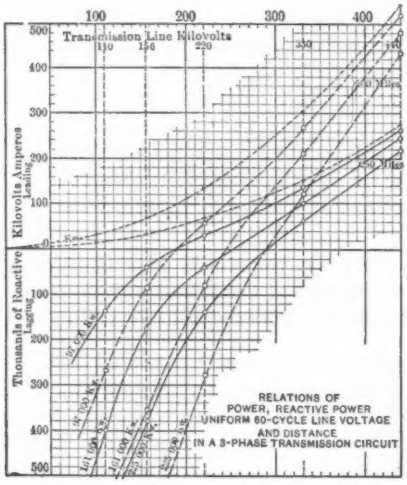


FIG. 26.

An open mind should be maintained toward the possible availability in the future, of high continuous voltages of 500 000 or more. Responsible manufacturers are engaged on the solution of this problem. They propose to use continuous high-voltage currents by rectifying alternating, three-phase, high-voltage currents at their source and resolving them into low-voltage, three-phase, alternating currents for distribution at their markets, with electron valves for each transformation. It does not appear likely that such continuous high-voltage currents will ever replace alternating high-voltage currents in the general power network. It does appear quite possible, however, that, in the future, they may be used for trunk-line connections between the large centers of power and the markets that have been consolidated by the general network.

The foregoing are the more important of the useful high-voltage actions found in the electrical power industry. There are also the non-useful or predacious high-voltage phenomena that occur in power-transmission circuits due to accidental causes. The causes of most predatory high-voltage phenomena are:

Over-voltages from within the transmission circuit, due to:

1.—Switching.

2.—Breaking short circuits.

3.—Arcs between circuit terminals and from some point in the circuit to the ground.

4.—Engine or turbine-generator runaways.

5.—Resonance.

Over-voltages from without the circuit, due to:

6.—Lightning, rarely from direct stroke, generally through the release of bound charges.

Corona formation, due to:

7.—Use of conductors that are too small to prevent over-stresses in the adjacent atmosphere, permitting an escape of power.

Conductor supports through:

8.—Depreciation and failure of line insulators.

Space factors causing:

9.—Flash-overs from conductors to towers, cross-arms or ground cables not due to over-voltages:

(*a*). Accounted for through tangible evidences, for example, such flash-overs may be caused by the surfaces of insulators being covered with salt-laden dust and, later, wetted with fog, mist, or rain, a circumstance that is likely to develop an accounted-for flash-over.

(*b*). Unaccounted-for, infrequent, and now under searching investigation.

Most of these enemy phenomena have been studied successfully, and their characters and the elimination of the troubles they cause are now thoroughly understood. Suppression of corona losses by conductors of proper dimensions, the nearly total suppression of flash-overs and short circuits by the proper construction of insulator and other conductor supports, and the high insulator durability have been accomplished. The causes of failure of transmission lines due to flash-over and other predatory phenomena listed, generally occur singly, both as to time and location. The power-service interruptions which they produce, have been reduced to a working minimum by means of double transmission lines and switching stations located at intervals of 25 to 50 miles and equipped with automatic switches that isolate the line section in trouble, and by care in construction, operation, and maintenance.

In regard to predatory high-voltage phenomena occurring in power transmission, much the same rule applies as in other lines of service, namely, eventually everything that can happen, will happen. For this reason, engineers responsible for the reliability of power service are devoting much study nowadays to every known form of electricity in action at high voltage, in order to establish a foundation on which to erect an understanding of the cause and prevention of all electrical accidents or failures that occur in power-transmission practice.

GROWTH OF THE USE OF ELECTRIC POWER IN SOUTHERN CALIFORNIA AND PROBABILITIES OF ITS FUTURE GROWTH WITH REFERENCE TO SOURCES OF HYDRAULIC POWER

By Harry W. Dennis,* M. Am. Soc. C. E., and H. A. Barre,† Esq.

Southern California is supplied with electric power by five companies, namely, the Southern California Edison Company, the Southern Sierras Power Company, City of Los Angeles, the Los Angeles Gas and Electric Corporation, and the San Diego Consolidated Gas and Electric Company. The last two companies depend on steam for primary power. The Edison Company serves a territory equal to the entire area of the States of Vermont, New Hampshire, Connecticut, Rhode Island, New Jersey, and Delaware, supplies 65% of the power used in Southern California, and must be depended on for 90% of the hydro-electric power yet to be developed inside the State for the use of Southern California. Therefore, a description of the growth of the Southern California Edison Company and the use to which its power is put, together with deductions from its extensive plans for future development, will be typical of Southern California.

It is interesting to note that, during the period from 1900 to 1920, the population represented by the ten counties served by the Edison Company has increased more than 300 per cent. Parallel with this phenomenal growth, the Edison Company has increased its generating capacity from 12 000 h.p., in 1900, to 380 000 h.p., in 1921. It has increased the number of kilowatt-hours generated from 379 900 000 in 1910 to 1 079 000 000 in 1921. The number of consumers during the same period has increased from 5 000 to 276 000. The details are given in Tables 11 to 15, inclusive.

TABLE 11.—Growth of Electric Power Production in Southern California, as Represented by the Southern California Edison Company.

Year.	Millions of Kilowatt-Hours Produced by:			Percentage, annual kilowatt-hour growth.
	Water power.	Steam and purchased power.	Total.	
1910	253.8	126.1	379.9	14.5
1911	279.3	162.9	442.2	16.2
1912	250.3	272.9	523.2	18.2
1913	281.5	309.3	590.8	13.0
1914	518.3	117.0	635.3	7.5
1915	555.9	103.7	659.6	4.0
1916	639.1	61.1	700.2	6.0
1917	608.6	151.7	746.8	6.2
1918	608.6	204.5	813.1	8.7
1919	564.9	361.2	926.1	14.0
1920	627.5	350.2	977.7	5.0
1921	832.6	246.5	1079.1	10.4

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TABLE 12.—Uses to Which Electrical Energy is Being Applied and Their Growth During the Period 1910 to 1921.

Figures Given in Millions of Kilowatt-Hours.

Year.	Total sold.	Lighting.	Agriculture.	Railroads.	Cement mills.	Industrial.
1910	327	37.3	73.3	189.7	.....	27.7
1911	363.8	44.1	72.0	207.6	8.9	31.2
1912	431.5	50.9	90.5	217.4	37.2	35.5
1913	490.9	59.1	111.0	236.7	42.8	41.3
1914	494.9	68.2	102.0	239.2	37.7	47.8
1915	513.7	74.4	107.0	248.6	30.4	53.3
1916	536.9	88.5	110.0	247.8	29.6	61.0
1917	631.5	86.9	129.0	262.3	36.9	116.4
1918	685.9	83.5	150.0	268.5	31.7	152.2
1919	759.6	94.9	162.0	252.3	28.0	222.4
1920	844.5	117.8	168.0	277.5	54.7	226.5
1921	917.7	137.6	176.6	271.4	54.4	277.7

TABLE 13.—Growth of Generating Station Capacity. This Included All the Systems Now Constituting the Southern California Edison Company.

Year.	Number of Plants Added:		Horse-power.
	Water power.	Steam power.	
1893-1900	5	1	12 000
1901-1906	7	3	41 600
1906-1907	1	1	34 400
1908-1910	1	1	55 000
1911-1914	2	1	149 000
1915-1917	1	.....	2 700
1918-1921	2	.....	94 500

TABLE 14.—Big Creek Power Project Power Plants, Ultimate Developments.

	Static head, in feet.	Installed horse-power.
Power House No. 1.....	2 130	86 000
Power House No. 2.....	1 860 and 2 400	387 000
Power House No. 3.....	855	200 000
Power House No. 4.....	340	67 000
Power House No. 5.....	1 480	134 000
Power House No. 6.....	1 650	147 000
Power House No. 7.....	1 900	40 000
Power House No. 8.....	729	180 000

Note.—There is now installed 43 000 h.p. at Power House No. 1; 65 000 h.p. at Power House No. 2; and 30 000 h.p. at Power House No. 8.

The five major uses to which this electrical energy is being applied are lighting, agriculture, railroads, cement mills, and manufacturing. The lighting use consists of municipal as well as private arc and incandescent lighting; the agricultural use consists chiefly of pumping for irrigation purposes; the railroads use more than 271 000 000 kw-hr. to operate the local city lines and interurban system, extending in all directions from Los Angeles; three cement mills account for a large amount of used power each year; and the manufacturing use includes more than eighty industries.

TABLE 15.—Big Creek Power Project Storage Reservoirs, Ultimate Development.

Name of reservoir.	Capacity, in acre-feet.	Total head, in feet, through which water may be used.
Vermilion Valley	90 000	5 914
Florence Lake	40 000	5 914
Blaney Meadows	15 000	5 914
Huntington Lake	88 000	5 914
Shaver Lake	146 000	4 324
Granite Creek	22 000	5 474
Jackass Creek	27 000	5 474
Chiquito Creek	18 000	3 574
Mammoth Pool	28 000	1 924
Total	474 000	.....

The relative magnitude of the different classes of business may be shown from the connected load for each class. The total connected load of the Edison Company is 800 000 h.p., and is subdivided as follows:

Lighting	280 000 h.p.
Agriculture	160 000 h.p.
Railroads	105 000 h.p.
Cement mills	25 000 h.p.
Manufacturing	230 000 h.p.

On account of the diversity in its use, this total connected load of 800 000 h.p. is served by a maximum simultaneous plant output of only 320 000 h.p. This diversity results from the alternating requirements of agriculture, industrial and domestic service, and the extent of the territory served.

Agriculture is bound to expand, and although the Edison Company supplies the power for irrigating 550 000 acres of land, there yet remain in the territory served more than 1 000 000 acres capable of being irrigated by pumped water. The 300 kw-hr. used yearly in irrigating 1 acre of arid land, raises the sale value of the land from $30 to $225 and yields an annual return of $50 to $100 in crops. Consideration of this fact leads to the conclusion that the use of electric power for irrigation requirements will expand until every available acre for which there is water will be brought into cultivation.

Modern conditions demand electric service for farm, home, and factory. Domestic consumption, which depends on population, will increase rapidly as long as new appliances continue to make power more and more a necessity in the home. Certain railroads in Southern California will be electrified when the increasing consumption of fuel oil again overtakes the supply.

Electricity is, therefore, the motive power on which the industries of California depend—agriculture, manufacturing, and transportation cannot go forward without an adequate supply of electric power. The demand for power increases in proportion to the growth and expansion of these industries.

Based on past performances in the growth of electric power production (Table 11), it appears that the rate has been an annually compounded increase of approximately 10 per cent. With a continuation of this rate of growth,

estimates of the load have been prepared, which indicate a total load in 1935 of more than 1 000 000 h.p. An additional capacity of about 50 000 h.p. per year is now required.

The water power plants of the San Antonio Light and Power Company, on San Antonio Creek, the Redlands Light and Power Company, on Mill Creek and the Santa Ana River, and the steam plant of the West Side Lighting Company, of Los Angeles, represent the beginning of the composite system now known as the Southern California Edison Company. These early plants, operating in the Nineties, were among the first in the generation of so-called high-tension transmission of power, not only in Southern California, but in the United States.

This harnessing of the small foot-hill streams comparatively close to Los Angeles, utilizing the water as it came, without storage, was the source, in the early days, of a power supply. As the load demands became larger, it was necessary to go farther away for the power until, in 1913, the initial installation of the well known Big Creek project was completed.

In order to meet the future demand, a comprehensive survey has been made of the potential power available and a construction program adopted covering the development of more than 1 260 000 h.p., 1 120 000 h.p. of which is on the San Joaquin River, 80 000 h.p. on the Kern River, and 21 000 h.p. on the Kaweah River. This program is expected to take care of the needs of the service for the next few years, and will be put into effect not only in the most economical way, but also in the order and in the necessary amounts which best supply the load demand.

This plan of construction is already in effect on the San Joaquin River and its tributary, Big Creek. The initial units were built on Big Creek, about 70 miles east of Fresno, Calif. These plants consisted of Huntington Lake Reservoir, with a storage capacity of 88 000 acre-ft., at an elevation of approximately 7 000 ft., supplying, successively, Big Creek Power Houses Nos. 1 and 2, each with an installed capacity of 43 000 h.p., operating under static heads of 2 130 ft. and 1 860 ft., respectively. The current was transmitted to Los Angeles, a distance of 241 miles, at 150 000 volts, over two, single-circuit, transmission lines, terminating in the large sub-station of Eagle Rock, near Los Angeles.

These power-houses and transmission lines were placed in operation in 1913, and represented an investment of about $16 500 000. This initial installation of 86 000 h.p. in itself was a large undertaking, but was only the beginning of the ultimate development of the San Joaquin River, which will be 1 250 000 h.p.

The general plan for developing this great source of hydro-electric power consists of the construction of a chain of power-houses in series between Huntington Lake, with an elevation of 7 000 ft., and the tail-water of Big Creek Power House No. 4, with an elevation of 1 000 ft. This chain of power-houses, in the order of their location, beginning at Huntington Lake, will consist of Power Houses Nos. 1, 2, 8, 3, and 4. Water from Huntington Lake may also be conducted to a proposed second storage reservoir known as Shaver

Lake, of 146 000 acre-ft. capacity, and in its passage between the two reservoirs, will go through Power House No. 5. From Shaver Lake, the water will pass through Power Houses Nos. 2, 8, 3, and 4.

In addition to these power houses using the stored waters of Huntington and Shaver Lakes, two other power sites, not using these stored waters, will be developed. One of these, known as Power House No. 7, located on the middle fork of the San Joaquin River, offers an ultimate development of 40 000 h.p., under a head of 1 900 ft. The other, known as Power House No. 6, located on the main San Joaquin River above its junction with Big Creek, is capable of a development of 147 000 h.p., at a head of 1 650 ft. The principal hydraulic features of the project are shown on Fig. 27.

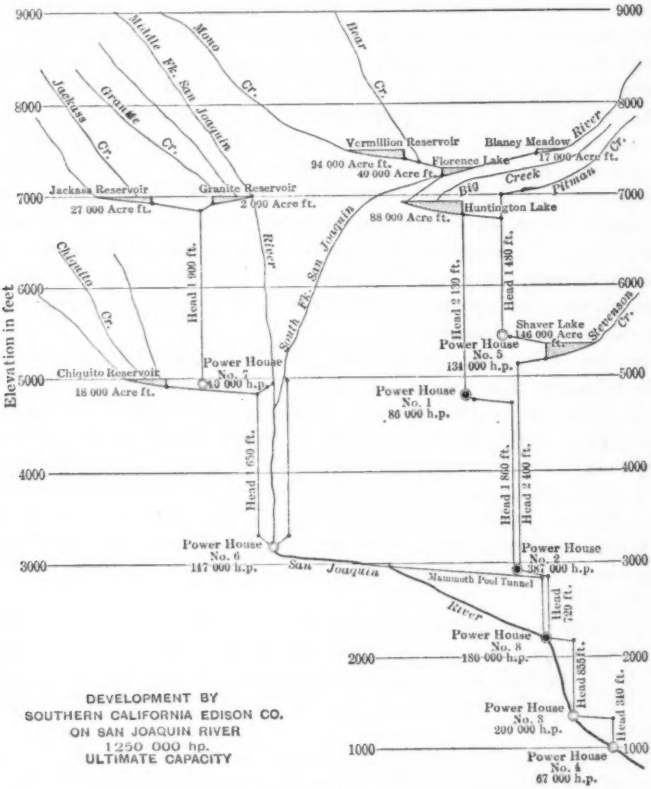


FIG. 27.

Huntington Lake is now supplied exclusively from Big Creek, with a drainage area of 79 sq. miles. In order to increase the water supply to Huntington Lake, the development program includes the construction of Florence Lake Tunnel, 13.5 miles in length, of a capacity of 1 000 sec-ft., which will

tap the Upper San Joaquin at Florence Lake and is expected to be completed by April, 1926. Through this tunnel, the waters of the South Fork of the San Joaquin River, together with Mono and Bear Creeks, will be diverted into Huntington Lake. Later, reservoir capacity up to 146 000 acre-ft. will be provided at Shaver Lake. A tunnel connecting Huntington Lake with Shaver Lake will be built, so that the combined capacities of those lakes can be used in storing the excess waters delivered through Florence Lake Tunnel. Complete utilization of this water will produce 750 000 000 kw-hr. per year, representing a value of more than $11 000 000 annually.

Following its proposed construction program, the Edison Company has completed the first 30 000-h.p. unit in Power House No. 8; added a third unit of 21 500 h.p. to Power House No. 2; raised the dam at Huntington Lake, and enlarged the capacity of the Eagle Rock Sub-Station. The investment in the San Joaquin River development has now increased $22 835 000, with an installation of 137 500 h.p.

Work is now in progress on Power House No. 3, which will have an initial capacity of 100 000 h.p. This plant will be completed in 1923. In addition, a third unit of 21 500 h.p. will be installed in Power House No. 1, and the voltage on the Big Creek transmission line will be increased to 220 000, thereby doubling its capacity.

The Edison Company will expend more than $27 000 000 during 1922 for new water-power plant construction and additional distribution facilities. Its plans call for the addition of 322 000 h.p. during the next 6 years, which will require an expenditure of more than $125 000 000 for its production and distribution.

In addition to the program of development of the Edison Company, the Southern Sierras Power Company and the City of Los Angeles also have developed water powers and plans for future water-power construction. The Los Angeles Gas and Electric Corporation and the San Diego Consolidated Gas and Electric Company, except for some small water-power projects, rely on steam-generated power for both present and future needs.

Among these central stations, the most important possibilities are those of the City of Los Angeles along its aqueduct. The ultimate capacity of the aqueduct plants is given as 250 000 h.p., of which 97 300 h.p. will have been placed in operation by the end of 1923.

As all the present plants of the City of Los Angeles are situated at hydraulic drops along the aqueduct, they utilize conduits which would be built for Water Department purposes even if no power was to be produced, and the minimum of hydraulic construction has been necessary. This fact, and the short distance of transmission, has resulted in the cheapest power that ever can be expected in Southern California, a condition from which the City of Los Angeles derives the full benefit. The City authorities have filed also on sites on King and Kern Rivers for a large amount of power, concerning which no published information is available.

As to future plants, it should be noted, both in connection with the Los Angeles Aqueduct and King and Kern Rivers, that the cost must include the full amount for the hydraulic construction and long-distance transmission.

The total power possibilities of the Southern Sierras Power Company aggregate about 175 000 h.p., of which approximately 75 000 h.p. is in operation delivering power in Nevada, Owens River Valley, Riverside, and San Bernardino Counties, and the Imperial Valley.

There remains as the future power supply for Southern California and, by releasing the San Joaquin power to the northern part of the State, for the whole of California, a pro rata part of the 4 500 000 h.p. on the Colorado River. Although much discussion has been given to this topic, there has not been time nor the necessity to make the careful investigation needed to prepare an adequate plan for utilizing the resources of the Colorado. There is a great temptation to start such a project on a scale that the available market cannot immediately support. What should possibly be looked for first is not how large a development can be made, but with how small a development can a start be made. Knowledge of this stream should be increased during the next few years, and the inception of a practicable plan devised for its initial and ultimate utilization, not on the basis of benefit to any one organization or interest, but in a manner whereby the public interest as a whole can most adequately be served.

The power industry has fully met the demands for electrical energy in Southern California from the beginning of commercial generation and transmission. The present companies in this field have definite, comprehensive plans to meet the increasing demand for a few years, and, at the appropriate time, must be prepared to undertake the development of the Colorado River for general service to the entire Southwest.

HYDRO-ELECTRIC POWER DEVELOPMENT AS RELATED TO THE ELECTRIFICATION OF RAILROADS

By C. F. Loweth,* M. Am. Soc. C. E.†

The title of this paper may lead to the assumption that cheap and abundant hydro-electric power in itself would justify the electrification of steam railroads. In the writer's opinion, such an assumption would be incorrect. The electrification of steam railroads, thus far, at least in the United States, has come about more largely from causes other than cheapness of electric power. The necessity of avoiding smoke and steam in tunnels and other restricted places and the need of increasing the capacity of tracks or other terminal facilities have generally been the determining factors which have brought about electrification. However, it is not improbable that conditions in this respect may change; doubtless, increase in volume of railway traffic and the known economies of electrification will tend to increase the use of electricity for the operation of railroads in localities where cheap and reliable hydro-electric power is available.

Something more than a mere showing of economy will be necessary to bring about a change from steam to electric operation of railroads. Electrification will involve a large increase of invested capital, and, at the same time, the retirement of many facilities which would not be needed with a change of power. Under the many adverse conditions with which the railroads in this country have contended in the past few years, their financial credit has been impaired; only a few of them have a surplus of earnings available for improvements, and the additional capital necessary therefor cannot be readily and cheaply obtained. Under the accounting rules of the Interstate Commerce Commission, the cost of facilities which are retired must be charged to profit and loss; such charges would be quite large, would be undesirable, and would have much the same effect as if they were charged to operating expenses.

The admitted economies of electrification have to contend with a constantly increasing efficiency of steam locomotives. Much study has been given to steam locomotive design with a view to greater efficiency; superheaters, stokers, better water, and many other appliances have made the modern steam locomotive a much more efficient and reliable prime mover than is generally supposed. This, together with the revision of gradients, additional and better facilities, and improved methods of operation, tends constantly to reduce the margin of economy which would justify electrification.

Electrification has some outstanding advantages which probably never can be equalled by steam operation. Some of these advantages are the longer continuous operating periods possible with electric locomotives, their greater power in single units, with less destructive effect on rolling stock and track than steam locomotives, the elimination of coal and water facilities, and the reduction in the number of engine terminals and the delays occasioned thereby.

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It is evident that these and all other advantages arising from electrification will vary with each particular railroad and for different points on the same railroad, and must be weighed as against all the factors affecting the cost of steam operation, of reducing gradients, and of increasing track and other facilities so as to cheapen present steam operation, and, lastly, the increased investment cost of electrification, including the retirement charges of facilities that would be retired. It is evident, therefore, that the cost of electric power is only one of many factors entering into the problem, but the more reliably and cheaply electric power becomes available, to that extent will it prove a favorable factor in electrification. To the extent that hydro-electric power is more readily available, more reliable, and cheaper than steam-generated power, to the same extent, approximately, will it be a more important factor. Even so, at this time it does not appear probable that the change from steam to electric operation would be justified for existing railroads, except where the volume of traffic is large, where it can be moved in large units for long distances, where fuel costs are high, and where operating costs due to adverse gradients and congestion of track and other facilities make steam operation somewhat more than ordinarily expensive.

Wherever the determining factor for the electrification of steam railroads has been the need of avoiding smoke in tunnels or densely populated terminals, or of increasing the capacity of tracks or other facilities, the insistence of the demand for the change has been so great, as a rule, that the cost of power has been only a minor factor. Generally, such cases have involved only restricted areas and have left much larger areas to continue to be operated by steam. The economies of electric operation in such cases probably would be increased materially by largely increasing the mileage of road electrically operated. The greater the extent of electrically operated mileage, the more important factor will become the cheapness of electric power.

Coal and fuel oil costs vary from time to time; in the last few years, the fluctuations have been extreme and, what is even more important and, at the same time, an indictment of American business and industrial methods, is the fact that fuel supplies have sometimes been uncertain. On the contrary, it would appear that, once the installation has been made, hydro-electric power would fluctuate as to supply within well defined limits, and that the cost, or at least the selling price, would be constant, or nearly so, for long periods. The ability to contract for an ample and reliable supply of hydro-electric power at favorable prices extending over a long term might easily prove a controlling factor in the electrification of railroads, especially for new railroads where the cost of coal, water, and engine terminal, as well as other facilities incident to steam operation, would be eliminated.

To illustrate this problem better, reference will be made to the electrification of sections of the Chicago, Milwaukee and St. Paul Railway, which is probably one of the largest electrification projects with respect to main-line mileage. Electrification was considered coincidently with the location and construction, in 1906-08, of the extension of this Railway System from Central South Dakota to the Pacific Coast, a distance of about 1 400 miles. The extension crossed several mountain ranges where long and heavy gradients

were necessary. Undeveloped water powers were abundant and power requirements for industrial uses were largely absent. These conditions led early to the suggestion that parts of the extension should be electrified; accordingly, the Railway Company acquired several water-power rights, and, at some places, located its lines with reference to the development of these powers. However, electrification was delayed, and on the completion of the railroad, it was operated by steam. It was not until several years later, about 1914, that the electrification of the section between Harlowton and Deer Lodge, Mont.—228 miles—was begun, and previous to the completion of this section, work was begun on the adjoining section from Deer Lodge to Avery, Idaho, making a total of 440 miles of main line. These sections crossed the main Continental Divide and the Belt and Bitter Root Mountain Ranges. Recently, the 209-mile section between Othello, in Central Washington, and Seattle and Tacoma, was electrified. A gap of main line between Avery and Othello (226 miles) is left, which undoubtedly would have been electrified before now, except for the abnormal conditions brought about by the World War and Federal control.

For various reasons, the Railway Company found it undesirable to add to and develop its own power requirements; therefore, it disposed of the power sites and made long-term contracts for power. The powers of which the Railway Company had obtained control were small compared with those now depended on, and the cost of their development would have been relatively much higher. They would have been subject to wide seasonal fluctuation as to quantity, and there would not have been the same high degree of reliability of power supply that exists under the present arrangement. The net cost of power to the Railway Company would also have been greater; at times, it would have been obliged to purchase power, and, at other times, it would have had surplus power to sell. The power now used and that already under contract for the 226-mile gap between Avery and Othello will be hydro-electric power. The contracts for power are with the Montana Power Company, the Inter-Mountain Power Company, the Washington Water Power Company, and the Puget Sound Light and Power Company, the first named company supplying the greater part. All these companies control large supplies of developed as well as of undeveloped water power. The contracts for this power run for long periods. The first contracts were for 99-year terms, and the later contracts will expire practically concurrently with those first made. The cost of the current delivered at the Railway Company's sub-stations is the same for all contracts, and at a very low price. The several power contracts are inter-related with agreements between the several power companies which, in effect, is practically equivalent to one contract covering all the railway's requirements for the electrified section. The Railway Company is, therefore, depending not on the individual power resources of these several companies, but on their combined resources, as the generating stations of the companies are so many and so completely tied together as to insure an uninterrupted supply for the entire electrified section. The eighteen or twenty generating stations supplying power under these contracts extend over a territory from Central Montana to Puget Sound, a distance of more than 650 miles,

with a north and south width of nearly 200 miles. The drainage areas supplying these powers not only cover a much larger territory, but they also differ greatly as to climatic and topographic conditions, a factor which it is believed insures a reliability of power supply under the most extreme conditions. The combined capacity of these stations is more than 360 000 kw. Within this area, and adjacent to it, are many undeveloped water powers. There can be no question but that this combination of ample, reliable, and cheap power, supplied under long-term contracts, at a low price, is an important factor in justifying this extensive railway electrification.

The other factors justifying the electrification were the remoteness from fuel supplies, the long stretches of steep gradients, the character of freight traffic, which permitted it to be concentrated in large train units, the considerable mileage of railway electrified, and last, but not least, the fact that the railway company would be able to control the flow of traffic over the electrified sections in such manner as largely to eliminate extreme peak loadings. All these last named considerations, of course, would apply more or less equally to an electrification using steam-generated current.

In this connection, it is interesting to note that this railway prior to electrification used oil for locomotive-fuel purposes exclusively for the main line and for the branches from Deer Lodge to the Pacific Coast, the oil being shipped by water from the Southern California fields. At that time and for a period of several years, oil was cheaper for the railway than coal, and had the further advantage of minimizing fire risks, an important consideration because the line crosses several forest reservations. Later, for those sections of the railway that were not electrified, it was necessary to change from fuel oil to coal.

Of the various electrifications of steam railroads in the United States, the majority are using steam-generated current. The several electrifications centering in and about New York City, the Pennsylvania Railway, at Philadelphia, Pa., and its West Jersey and Sea Shore Railway, the Norfolk and Western Railway, the Detroit, Sarnia, Hoosac, and Baltimore Tunnels are all operated by steam-generated current. Both steam-generated and hydro-electric currents are used for the Canadian National Railway Tunnel at Montreal, Que., Canada, mostly the latter, whereas for the Great Northern Railway Tunnel in Washington only hydro-electric current is used. The Butte, Anaconda and Pacific Railway (which carries a heavy freight traffic) and the Spokane and Inland Empire Railroad lines use hydro-electric power exclusively, the former being supplied by the Montana Power Company.

The abundance and cheapness of hydro-electric power, especially as compared with steam-generated power, have probably been more the controlling factors in the electrification of steam railroads in foreign countries than in the United States. This is shown in the official reports made to the International Railway Congress held in Rome, Italy, in the early part of 1922. As to Sweden and Norway, Mr. Ofverholm, Chief of the Electrical Department of the Swedish State Railways, and Official Reporter to the Congress, states:

"Sweden, Norway, and Denmark have not the necessary quantity of home-produced coal. The coal fields that exist in the south of Sweden are quite inadequate, and the coal is of poor quality. On the other hand, Sweden and Norway have plenty of water power, though this is not the case with Denmark. In the first two countries, railway electrification on a large scale has been contemplated, in order to utilize water power instead of depending on imported coal. In Denmark, owing to the lack of water power, no such schemes are in existence.

"The desire to make the railways independent of foreign coal has been entertained in Sweden for a long time."

Later in the report, where the program for electrification is being considered, he states:

"The chief point is to make the railways independent of foreign coal; therefore, the lines that have the highest coal consumption should be the first to be electrified. * * * The research work carried out in connection with the electrification has included an investigation of the total amount of hydraulic power available."

The reporter for Italy, Mr. Alfredo Donati, states:

"The new program for the extension of electric traction in Italy which was drawn up by the Administration of the Italian State Railways in common with the Government, anticipates the electrification of about 2 800 miles of railway which has been selected with care from amongst those which, owing to their gradients and their heavy traffic, now consume the greatest quantity of coal."

It was stated that this program when completed will effect an annual saving of about 1 250 000 tons of coal and a consumption of about 600 000 000 kw-hr. of current per annum, the greater part of which will be generated by water power. This program, however, is not altogether to save coal, for, later, it is stated that, on certain of the lines to be electrified, the limit of traffic which could be carried by steam traction, had been attained, and electrification had become necessary for increasing the carrying capacity of the lines; also, that, on a number of Italian lines, electrification has been hastened on account of the difficulties of steam operation through the numerous tunnels.

The Federal Railway Commission of Switzerland, reporting to the International Congress, concludes with the following statement:

"The electrification of the Federal Railways is an economic necessity, and ought to be brought about if regular working at all times and under all conditions is to be assured. We must not forever remain dependent on foreign countries, which may withhold their coal, or at any rate dictate their prices without consideration for us. Of the total imports in 1920 (approximately 991 700 ton), 303 914 ton, or 30.5%, came from England, and 553 986 ton, or 55.9%, from America, while our former principal sources, Belgium, the Sarre Territory, and the Ruhr Valley, only delivered to us altogether 65 500 ton, or 6.6 per cent. This fact shows on what casual conditions our coal supply and, consequently, the working of our railways, depends, without taking into account the disturbing factors arising out of strikes, transport difficulties, etc."

France, according to M. Sabouret, Chief Engineer, Technical Department of the Orleans Railway, Reporter to the International Congress, has an ambitious electrification program. In addition to some electrification already completed, and the program for the State-owned railways, the Orleans Rail-

way Company has a program for the electrification of 1 240 miles of railroad, the Midi Railway for 1 680 miles, and the Paris-Lyons-Mediterranean Company for a considerable mileage.

The Orleans Railway will use largely hydro-electric power supplemented with some steam-generated current from a station near Paris; the Midi Railway will use largely, if not exclusively, hydro-electric power; the Paris-Lyons-Mediterranean Company will obtain its principal sources of current from hydro-electric plants on the Upper and Middle Rhône River. All three railways will develop their own power, and, in some cases, the developments will be made jointly with the State-owned railways. These three railways have under consideration a plan for the joint development of hydro-electric power on the Truyère River, in which the development would exceed 200 000 h. p., the supply being exceptionally regular. The plan is to divide the power between them in such a manner as to supplement for each railway the minimum power from its own stations during periods of low water. The reporter says:

"The flow of the watercourses of the Central Plateau is complementary to those of the Alps and Pyrenees; the Truyère will feed the Rhône and Garonne basins during the winter, and the Central Region during the summer. This large programme of the organization of hydro-electric energy shows clearly the obligation imposed on the three companies of associating themselves in a work of unification required in the National interest and to promote the linking up of the three great producing districts."

These references are illustrative of the development of the electrification of railroads in other countries; they appear to indicate the probability that hydro-electric power, by virtue of its cheapness and other qualities, has been more of a factor in extending electrification in foreign countries than it has been in the United States.

The Pacific Coast, with its vast hydro-electric power potentialities, seems to present favorable conditions for the electrification of steam railroads. The power possibilities are probably in excess of commercial and industrial needs and are likely so to continue, whereas coal and fuel oil supplies are becoming more costly and less certain. The topography of much of the country traversed by the railroads and the extent and character of the traffic result in railroad-operating conditions which probably could be met more economically by electrification than by steam.

In the case of new railroad construction, assuming that, in spite of excessive and oftentimes conflicting regulation of the railroads by Federal and State authorities, capital for new railroad construction can be found, electrification would offer advantages for such lines as have promise of a reasonable volume of traffic. The saving in first cost of water and fuel facilities and in the fewer engine terminals required would be considerable, and it is probable that the location of the line and the ruling gradients adopted would be such as to reduce the cost of construction as compared with a road intended to be operated by steam.

It is possible that the requirements of the Federal Water Power Act may make it impossible to contract for power for periods longer than fifty years at

the most, and together with State regulation of public utility rates, may make uncertain the permanency or uniformity of power rates extending over long periods; if so, the progress of railroad electrification may be somewhat retarded.

The subject of this paper has been covered in such a brief and general manner that no conclusions are warranted. However, it would seem reasonably safe to assume that hydro-electric power would be an important factor in steam-railway electrification under the following conditions:

(*a*).—When cheap and available over such an extended area as would include a large main-track mileage for the individual railroad under consideration.

(*b*).—When supplied from several sources tied together so as to insure reliability.

(*c*).—When it can be contracted for long periods.

(*d*).—When its cost reduced to terms of locomotive tractive effort, is cheaper than that of steam-generated power.

WATER POWER POTENTIALITIES OF THE PACIFIC COAST

By Frederick Hall Fowler,* Assoc. M. Am. Soc. C. E.

Synopsis

This paper approaches the water-power potentialities of the Pacific Coast from the standpoint of market demand for their future development. It gives the outputs from water-power and fuel-burning plants in the seven States within the Pacific drainage for 1920 and 1921 and, from more extensive records, it computes the rate of growth in the seven principal market centers within this area during the last decade and forecasts the loads in the same markets for 1930 and 1940. These market demands are shown to scale on a map (Fig. 28) of the Pacific drainage, on which also are shown, to the same scale, the flat load power capacities and approximate locations of many of the more important undeveloped water-power projects or sites. The general conditions on the principal water-power streams are outlined in the text.

The paper is not intended as a complete inventory of all undeveloped water powers of the Pacific drainage, but the study has been carried far enough to show the general relation between present and future demand and supply. No attempt has been made to cover the market in the relatively small parts of Montana, Wyoming, Colorado, and New Mexico within the Pacific drainage.

Market Conditions and Growth of Load

Interest in potential water power is measured largely by present and prospective power needs and the relative expense of producing energy by various well established methods. If in any region there is an urgent present and prospective power demand, some source of energy, usually that which at the time is cheapest, will be developed. It may turn out later that the original conditions are changing and to the extent of that change, the trend of development will change. It is reasonable, therefore, to consider the potential water powers in the streams west of the Continental Divide in relation to the demand for their utilization in competition with power developed by fuel.

Four types of fuel have been used within this territory for the commercial generation of power: Oil, coal, natural gas, and wood, listed in the order of magnitude of their use.

Oil, supplied chiefly from California fields, is practically the only fuel utilized for the development of power in steam-electric stations over a large area in the Western States. The greatest use of fuel oil is in California where 5 600 000 bbl. were burned in 1920 and 3 300 000 bbl. in 1921. It is used to a more limited extent in Arizona, Washington, and Oregon. Immediately after the World War, when the price of fuel oil advanced rapidly, there was a tendency for Colorado and Wyoming coal to be shipped westward and to displace the use of oil on the railroads, but only a long continued rise in the price of oil would displace oil in steam stand-by stations on the hydro-electric systems.

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A readily available supply of good steam coal is found in only limited areas west of the Rocky Mountains. It so happens, however, that in the areas where coal is found, water power is also abundant and, for the West, unusually cheap. It is, therefore, a strong competitor for the power market. Washington is the only one of the seven States in which coal has been used to any great extent for the generation of electric power.

Increased labor costs and difficulties of mining coal and the growing demand for oil for purposes other than fuel (coupled with the marked tendency of every oil field to drop rapidly in rate of production from the day that it is first brought in) will doubtless put these two sources of energy at an increased disadvantage as primary sources of power.

Natural gas is available in only a few comparatively restricted areas in California. The most important area of this type is in the southwestern side of the San Joaquin Valley where one of the operating electric companies is now generating steam power from gas, not only for use in its own system, but for transmission southward to the companies serving the power market in and around Los Angeles.

Wood has been used for fuel in electric generation only to consume the waste of some of the largest lumber mills in Oregon, Washington, California, and Idaho.

The relative importance of the various types of fuel may be seen by reference to Table 16 which is transcribed from the general compilations of the U. S. Geological Survey.

In the future, as in the present and past, steam stand-by stations using oil or coal for fuel will play an important part in the hydro-electric systems of the West, but from present indications, a constantly decreasing percentage of the total output from steam power and an increasing percentage from water power may be expected.

Most of these facts have long been common knowledge to those directly engaged in Western water-power development, but are presented here for the benefit of those who have not been in close contact with Western power problems.

Few persons realize that even now inter-connecting transmission systems extend, except for minor breaks, from Eastern Idaho and Central Utah westward and northward through the Columbia and Snake River Valleys to the Pacific Coast in the Puget Sound region and thence southward through Washington, Oregon, and California, to the Mexican line and Western Arizona. The lines in west Central Nevada, now isolated, are soon to be connected with Central California, and short connections only will be necessary to unite the systems of Southern California and Arizona, Utah and Colorado, Idaho and Eastern Montana, and Idaho and Oregon. On the completion of these ties the entire inhabited country of the Pacific drainage will be served by an interconnecting network. Some of the present inter-connections are weak, but they may be strengthened at relatively small cost as market conditions warrant. The large investment in existing systems will naturally have a marked effect on the order of development of new projects.

TABLE 16.—PRODUCTION OF ELECTRIC POWER AND CONSUMPTION OF FUELS BY PUBLIC UTILITY POWER PLANTS IN THE SEVEN STATES ENTIRELY WITHIN THE PACIFIC DRAINAGE, 1920 AND 1921, REPORTED BY U. S. GEOLOGICAL SURVEY.

	Total Power Produced.		Produced by Water Power.			Produced by Fuel Power.			Coal Consumed.		Fuel Oil Consumed.		Gas Consumed.		Kilowatt-hours produced by wood fuel.
	Kilowatt-hours.	Percentage of United States total.	Kilowatt-hours.	Percentage of State total.	Percentage of United States total.	Kilowatt-hours.	Percentage of State total.	Percentage of United States total.	Short tons.	Percentage of United States total.	Barrels.	Percentage of United States total.	Thousands of cubic feet.	Percentage of United States total.	
1920															
Arizona	159 228 000	0.37	106 527 000	66.9	0.66	52 701 000	33.1	0.19	5 579	0.01	238 517	1.82	0	.....	.....
California	3 735 645 000	8.58	2 568 541 000	68.8	15.91	1 167 104 000	31.2	4.26	0	0.00	5 646 085	43.02	2 970 956	12.03	6 671 000
Idaho	590 864 000	1.37	588 978 000	99.7	3.64	1 886 000	0.3	0.01	435	0.00	40	0.00	0	.....	1 284 000
Nevada	33 549 000	0.08	32 690 000	97.4	0.20	859 000	2.6	0.00	2 847	0.01	2 937	0.02	0	.....	0 000
Oregon	475 543 000	1.09	370 867 000	78.0	2.29	104 676 000	22.0	0.38	2 779	0.01	24 052	0.18	0	.....	99 972 000
Utah	212 321 000	0.49	212 243 000	100.0	1.31	78 000	0.0	0.00	212	0.00	0	0.00	0	.....	0 000
Washington	1 196 725 000	2.75	1 154 135 000	96.5	7.14	42 590 000	3.5	0.16	43 891	0.12	141 523	1.08	0	.....	9 888 000
1920	6 403 875 000	14.73	5 033 981 000	78.6*	31.15	1 369 894 000	21.4*	5.00	55 743	0.15	6 053 104	46.12	2 970 956	12.03	117 815 000
1921															
Arizona	132 484 000	0.32	93 120 000	70.3	0.62	39 314 000	29.7	0.15	2 606	0.01	180 406	1.50	0	0.00	.....
California	3 982 988 000	9.72	3 227 261 000	81.0	21.55	755 677 000	19.0	2.90	0	0.00	3 343 269	27.76	3 167 454	13.35	7 269 000
Idaho	550 149 000	1.34	546 896 000	99.4	3.65	3 253 000†	0.6	0.01	0	0.00	0	0.00	0	0.00	3 253 000
Nevada	36 532 000	0.09	36 139 000	98.9	0.25	393 000	1.1	0.00	1 885	0.01	593	0.00	0	0.00	0 000
Oregon	468 534 000	1.14	374 065 000	79.9	2.50	94 469 000	20.1	0.36	1 165	0.00	36 613	0.30	0	0.00	84 156 000
Utah	119 551 000	0.29	119 532 000	100.0	0.80	19 000	0.0	0.00	56	0.00	0	0.00	0	0.00	0 000
Washington	1 176 662 000	2.87	1 139 909 000	96.9	7.61	36 753 000	3.1	0.14	31 695	0.10	103 889	0.86	0	0.00	12 357 000
1921	6 466 800 000	15.77	5 536 922 000	85.62*	36.98	929 878 000	14.38*	3.56	37 407	0.12	3 664 770	30.42	3 167 454	13.35	107 035 000

* Percentage of total for the group of seven States.

† Produced by plants using wood fuel.

As a basis for comparing the load carried by the systems serving the Pacific drainage with the total load for the United States, the figures of the U. S. Geological Survey are of great value. For 1921, the survey reported 40 976 000 000 kw-hr. for the entire country, of which 14 970 000 000 kw-hr., or 36.5%, was produced by water power, and the remainder by fuel. The total power output of the seven States (Arizona, California, Idaho, Nevada, Oregon, Utah, and Washington) entirely within the Pacific drainage was 6 467 000 000 kw-hr., or 15.77% of the total for the country, and the water power output in these seven States was 5 537 000 000 kw-hr., or 85.62% of the total production within the group of States, and 36.98% of the water-power output for the entire country. The detailed figures by States for the group are give in Table 16.

The outputs reported by the U. S. Geological Survey check closely the 1920 and 1921 outputs available from other sources, and these independent records extend back for a full decade; they are in varying detail, in different sections of the West, but in all sections are sufficiently complete and accurate to furnish a basis for computations of past growth in demand for power, and afford also the only logical basis for forecasting the loads of the future. On the rate of growth of these future loads will depend the demand for the potential water powers. The following paragraphs outline the past growth in output, and on Fig. 28 are shown in simple graphic form:

(*a*) The actual load at seven given market centers in 1921 and the estimated loads at the same points in 1930 and 1940; and
(*b*) Some of the important undeveloped powers that are now scheduled for construction by public utilities and municipalities or that have been investigated in more or less detail by Federal or State authorities.

Both loads and power capacities are shown on the same scale so that they may be readily compared, and arcs have been described around the various load centers to show the approximate transmission distances.

This diagram is not intended as a catalogue of potential power, and a short study will show that many important sites that have not been fully investigated, or for which there is small prospective demand, have been omitted. Enough power possibilities have been recorded, however, to show several outstanding facts, among which may be mentioned the wonderful potential powers of the Columbia River (so large in many instances as not to be economically feasible unless new industries are established to utilize their output); the importance of the California sites still remaining undeveloped; and the availability of the southernmost of these sites in competition with the principal sites on the Colorado, that have recently received much publicity.

In some instances, the so-called "markets" are definite centers and, in others, are broad territories. They may be classified, or described, as follows.

Puget Sound Market.—This market includes the territory on the shores of Puget Sound and eastward to the crest of the Cascade Range, from the Canadian line southward to the Columbia water-shed, with Seattle, Wash., as the principal load center. The combined output of the most important municipal and public utility systems serving this territory shows an increase from 238 600 000 kw-hr. in 1911 to 716 400 000 kw-hr. in 1921 and, approximately, 750 000 000 kw-hr. for 1922; the rate of growth from 1911 to 1922, therefore, was

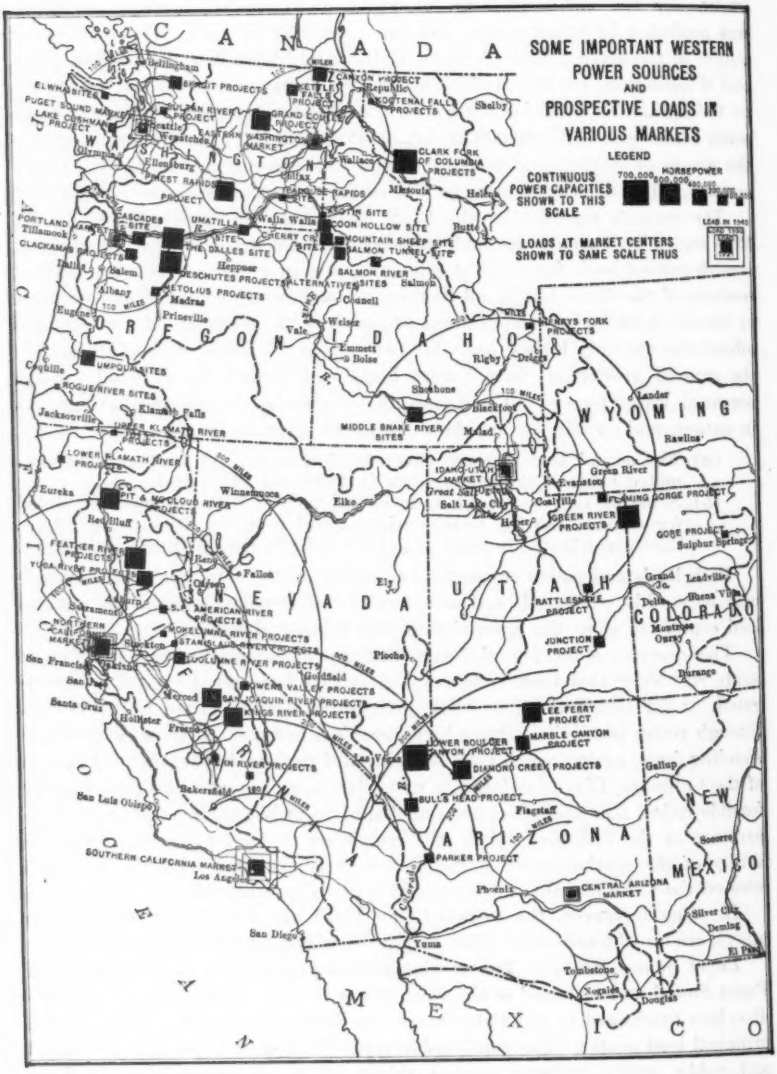

SOME IMPORTANT WESTERN POWER SOURCES AND PROSPECTIVE LOADS IN VARIOUS MARKETS
LEGEND
CONTINUOUS POWER CAPACITIES SHOWN TO THIS SCALE
HORSEPOWER
LOADS AT MARKET CENTERS SHOWN TO SAME SCALE THUS
LOAD IN 1940
C A N A D A
O R E G O N
I D A H O
W Y O M I N G
N E V A D A
U T A H
C O L O R A D O
A R I Z O N A
N E W M E X I C O
M E X I C O
Bellingham
ELWHA SITES
PUGET SOUND MARKET
LAKE CHELAN PROJECT
Seattle
Wenatchee
Ellensburg
Yakima
Walla Walla
Lewiston
Spokane
Missoula
Helena
Butte
Shelby
Havre
Portland
Tillamook
CLACKAMAS PROJECTS
Salem
Albany
Eugene
Madras
Prineville
Vale
Weiser
Emmett
Boise
Nampa
Caldwell
Payette
Ontario
Baker
La Grande
Pendleton
Hermiston
The Dalles
Hood River
Bend
Klamath Falls
Roseburg
Coquille
Jacksonville
Eureka
Red Bluff
Redding
Oroville
Sacramento
Stockton
Modesto
Merced
Fresno
Hollister
San Jose
San Francisco
Santa Cruz
Bakersfield
San Luis Obispo
Los Angeles
San Diego
Yuma
Phoenix
Tucson
Nogales
Douglas
Tombstone
Deming
Silver City
El Paso
Gallup
Flagstaff
Winnemucca
Elko
Ely
Wells
Ogden
Salt Lake City
Provo
Logan
Pocatello
Idaho Falls
Blackfoot
Shoshone
Twin Falls
Rupert
Malad
Evanston
Green River
Rawlins
Lander
Casper
Cheyenne
Fort Collins
Denver
Colorado Springs
Pueblo
Trinidad
Durango
Montrose
Grand Junction
Delta
Leadville
Grand Jct.
Buena Vista
Sulphur Springs
Gore Project
Flaming Gorge Project
Green River Projects
Coalville
Junction Project
Rattlesnake Project
Lee Ferry Project
Marble Canyon Project
Lower Boulder Canyon Project
Diamond Creek Projects
Bulls Head Project
Parker Project
Central Arizona Market
Southern California Market
Great Salt Lake
Idaho-Utah Market
Middle Snake River Sites
Grand Coulee Project
Kettle Falls Project
Kootenai Falls Projects
Clark Fork of Columbia Projects
Cabinet Projects
Spokane Rapids
Coeur d'Alene Lake
Priest Rapids Project
Cascades Site
Umatilla Site
Cherry Creek Site
The Dalles Site
Hoover Site
Deschutes Projects
Alternative Sites
Metolius Projects
Umpqua Sites
Rogue River Sites
Klamath River Projects
Lower Klamath Project
Pit & McCloud River Projects
Feather River Projects
Yuba River Projects
American River Projects
Mokelumne River Projects
Stanislaus River Projects
Tuolumne River Projects
Merced Falls
Bowman Valley Projects
San Joaquin River Projects
Kings River Projects
Kern River Projects
Coon Hollow Site
Mountain Sheep Site
Salmon Tunnel Site
Salmon River Sites
Salmon
Challis
Sawtooth Site
Big Wood River Projects
Henrys Fork Projects
Bear River Projects
Weber River Projects
Provo River Projects
Jordan River Projects
Ogden River Projects
Logan River Projects
Bear Lake Projects
Snake River Projects
Teton River Projects
Portneuf River Projects
Raft River Projects
Owyhee River Projects
Malheur River Projects
Powder River Projects
Burnt River Projects
Grande Ronde River Projects
Imnaha River Projects
Snake River Projects
Clearwater River Projects
Palouse River Projects
Spokane River Projects
Pend Oreille River Projects
Flathead River Projects
Kootenai River Projects
Kootenai Falls Projects
Clark Fork of Columbia Projects
Missoula
Helena
Butte
Shelby
Havre
Great Falls
Lewistown
Billings
Miles City
Glendive
Williston
Minot
Bismarck
Mandan
Dickinson
Aberdeen
Pierre
Rapid City
Sioux Falls
Yankton
Sioux City
Omaha
Lincoln
Grand Island
Hastings
Kearney
North Platte
Scottsbluff
Alliance
Chadron
Valentine
Norfolk
Fremont
Omaha
Council Bluffs
Des Moines
Davenport
Rock Island
Peoria
Springfield
Quincy
Hannibal
St. Louis
Kansas City
Topeka
Wichita
Hutchinson
Salina
Dodge City
Garden City
Liberal
Amarillo
Lubbock
Abilene
Fort Worth
Dallas
Waco
Austin
San Antonio
Houston
Galveston
Beaumont
Port Arthur
Lake Charles
New Orleans
Baton Rouge
Shreveport
Monroe
Vicksburg
Jackson
Meridian
Tuscaloosa
Birmingham
Montgomery
Mobile
Pensacola
Tallahassee
Jacksonville
Orlando
Tampa
Miami
Key West
Atlanta
Augusta
Savannah
Charleston
Columbia
Greenville
Asheville
Knoxville
Nashville
Memphis
Louisville
Cincinnati
Columbus
Dayton
Toledo
Cleveland
Akron
Youngstown
Pittsburgh
Erie
Buffalo
Rochester
Syracuse
Albany
Utica
Binghamton
Scranton
Wilkes-Barre
Harrisburg
Philadelphia
Trenton
Newark
New York
Jersey City
Hartford
New Haven
Providence
Boston
Worcester
Springfield
Portland
Bangor
Augusta
Concord
Manchester
Burlington
Montpelier
Rutland
Albany
Troy
Schenectady
Utica
Syracuse
Rochester
Buffalo
Niagara Falls
Toronto
Hamilton
London
Windsor
Detroit
Lansing
Grand Rapids
Kalamazoo
Battle Creek
Jackson
Ann Arbor
Flint
Saginaw
Bay City
Muskegon
Milwaukee
Racine
Kenosha
Waukegan
Chicago
Gary
South Bend
Fort Wayne
Indianapolis
Terre Haute
Evansville
Peoria
Springfield
Decatur
Champaign
Danville
Bloomington
Rockford
Freeport
Dubuque
Davenport
Rock Island
Moline
Galesburg
Quincy
Hannibal
St. Louis
East St. Louis
Alton
Belleville
Carbondale
Cairo
Paducah
Hopkinsville
Clarksville
Nashville
Chattanooga
Knoxville
Asheville
Greenville
Spartanburg
Columbia
Charleston
Savannah
Augusta
Atlanta
Macon
Columbus
Montgomery
Mobile
Pensacola
Tallahassee
Jacksonville
Orlando
Tampa
Miami
Key West
Gulf of Mexico
Atlantic Ocean
Pacific Ocean
Hudson Bay
Lake Superior
Lake Michigan
Lake Huron
Lake Erie
Lake Ontario
Gulf of St. Lawrence
Nova Scotia
New Brunswick
Prince Edward Island
Quebec
Ontario
Manitoba
Saskatchewan
Alberta
British Columbia
Yukon
Northwest Territories
Alaska
Hawaii
Puerto Rico
Cuba
Jamaica
Haiti
Dominican Republic
Bahamas
Bermuda
Panama Canal
Panama
Costa Rica
Nicaragua
Honduras
Guatemala
Belize
Yucatan
Campeche
Tabasco
Veracruz
Tamaulipas
Nuevo Leon
Coahuila
Chihuahua
Sonora
Sinaloa
Durango
Zacatecas
San Luis Potosi
Tamaulipas
Nuevo Leon
Coahuila
Chihuahua
Sonora
Baja California
Guadalajara
Mexico City
Puebla
Oaxaca
Chiapas
Tehuantepec
Isthmus of Tehuantepec

FIG. 28.

11% compounded annually. Future growth has been forecasted on the following decreasing rates compounded annually: 1922 to 1925, 11%; 1926 to 1930, 10%; 1931 to 1935, 9%; and 1936 to 1940, 8 per cent. The following actual load for 1921, and the estimated loads for 1930 and 1940, are plotted on Fig. 28:

Year.	Kilowatt-hours.	Equivalent mean horse-power.
1921	716 400 000	109 600
1930	1 656 700 000	253 400
1940	3 745 500 000	572 900

Eastern Washington Market.—This market includes the central part of the Columbia Basin and the territory eastward to the State boundary, with the principal load center at Spokane, Wash. The combined output of the most important public utility systems shows an increase from 150 600 000 kw-hr. in 1910 to 430 200 000 kw-hr. in 1921 and about 442 600 000 kw-hr. in 1922; the rate of growth from 1910 to 1922, therefore, was 9.5% compounded. Further growth has been forecasted on the following decreasing rates compounded annually: 1922 to 1925, 9%; 1926 to 1930, 8%; 1931 to 1935, 7%; and 1936 to 1940, 6 per cent. The following actual load for 1921 and the estimated loads for 1930 and 1940, are plotted on Fig. 28:

Year.	Kilowatt-hours.	Equivalent mean horse-power.
1921	430 200 000	65 800
1930	844 600 000	129 200
1940	1 585 400 000	242 500

Idaho-Utah Market.—This territory includes the Snake River Valley in Southern Idaho, from the western nearly to the eastern boundary of the State, and the Ogden and Salt Lake City regions in Utah. To date, there has been a break in the center of the territory (between two divisions of a single system), and the inclusion of the entire territory in one market with a load center north of Ogden is purely arbitrary; lines that will actually consolidate the system are now contemplated, however, so that the arbitrary classification seems to be warranted. The combined output of the most important public utility systems serving the market shows an increase from 188 000 000 kw-hr. in 1910 to 316 900 000 kw-hr. in 1914, 670 200 000 in 1921, and about 641 600 000 in 1922. The growth from 1914 to 1921 was at the rate of 11% compounded, with wide variation from the curve in certain years, due probably to fluctuations in mining demand. Future growth has been forecasted, starting with the actual output of 1921, on the following decreasing rates compounded annually: 1921 to 1925, 10%; 1926 to 1930, 9%; 1931 to 1935, 8%; and 1936 to 1940, 7 per cent. The following actual load for 1921 and the estimated loads for 1930 and 1940, are plotted on Fig. 28:

Year.	Kilowatt-hours.	Equivalent mean horse-power.
1921	670 200 000	102 500
1930	1 509 800 000	230 900
1940	3 112 300 000	476 100

Portland Market.—This market includes the territory in Northwestern Oregon and Southwestern Washington having a definite commercial and load center at Portland, Ore. The combined output of the most important public utility systems shows an increase from 152 200 000 kw-hr. in 1911 to 411 500 000 kw-hr. in 1921, the rate of growth during the period, therefore, was 10.5% compounded annually. Further growth has been forecasted on the following decreasing rates compounded annually: 1921 to 1925, 10%; 1926 to 1930, 9%; 1931 to 1935, 8%; and 1936 to 1940, 7 per cent. The following actual load for 1921, and the estimated loads for 1930 and 1940, are plotted on Fig. 28:

Year.	Kilowatt-hours.	Equivalent mean horse-power.
1921	411 500 000	62 900
1930	927 000 000	141 800
1940	1 910 400 000	292 200

Northern California Market.—This market includes the populated territory in California north of Monterey on the coast and Merced in the Central Valley, and, in addition, the territory in and around Reno in Western Nevada, which is to be joined to the California systems. The networks of the operating companies extend over a wide area, but a combined load center may be fixed in the vicinity of Port Costa on the eastern side of San Francisco Bay.

The combined output of practically all public utilities and municipal systems shows a uniform growth from 1 241 400 000 kw-hr. in 1914 to 1 999 800 000 kw-hr. in 1920, and a falling off to 1 986 600 000 kw-hr. in 1921; the rate of growth from 1914 to 1921 was 7% compounded. Future growth has been forecasted on the following decreasing rates compounded annually: 1921 to 1925, 7%; 1926 to 1930, 6%; and 1931 to 1940, 5 per cent. It will be noted that the estimated percentage was not decreased in 1936 as in other estimates; this is because the rate was already very low. The following actual load for 1921 and estimated loads for 1930 and 1940 are plotted on Fig. 28:

Year.	Kilowatt-hours.	Equivalent mean horse-power.
1921	1 986 600 000	303 900
1930	3 485 400 000	533 200
1940	5 677 500 000	868 500

Southern California Market.—This market includes the populated territory in California south of Monterey on the coast, Merced in the interior valley, and Mono Lake Basin east of the Sierra, as well as the mining region of Southwestern Nevada. The chief load center is in the vicinity of Los Angeles, but a secondary center of great importance is in the San Joaquin Valley; nevertheless, on the map (Fig. 28), the entire load for the market is centered at Los Angeles.

The combined output of practically all public utility and municipal systems shows a steady growth from 854 100 000 kw-hr. in 1913 to 1 025 800 000 kw-hr. in 1916, and then a steady growth, at an increased rate, to 2 002 600 000 kw-hr. in 1921. The rate of growth from 1916 to 1921, 14.4% compounded annually,

is phenomenal. Future growth has been forecasted on the following decreasing rates compounded annually: 1921 to 1925, 13%; 1926 to 1930, 12%; 1931 to 1935, 10%; and 1936 to 1940, 8 per cent. Objection may be raised to the fairness of decreasing the rate in this estimate by 2% in each of the last three 5-year periods, instead of 1% every 5 years, as in other estimates (or even maintaining a flat rate for the last 10 years, as in the Northern California market). Several factors demand a more rapid rate of decrease: (*a*) the astonishing rate of growth has been maintained only since 1916; (*b*) the cumulative effect of compounding at so high a rate would pre-suppose a rate of growth in population and industry that has never been equalled for any very long period; and (*c*) the rapidly developing territory in the Central San Joaquin Valley, that might well be classified with the Northern California market (and has actually transmitted power to the northern market) has contributed to the high rate of growth recorded for the Southern California market. For these reasons, the speaker believes the sliding scale adopted is, if anything, more than ample. The following actual load for 1921, and estimated loads for 1930 and 1940 are plotted on Fig. 28:

Year.	Kilowatt-hours.	Equivalent mean horse-power.
1921	2 002 600 000	306 300
1930	5 754 300 000	880 200
1940	13 616 800 000	2 083 000

Central Arizona Market.—This market includes the mining and agricultural territory in Arizona, extending from Prescott and Jerome southward to Phœnix, and thence eastward to Miami, Globe, and Clifton. Part of the mining load in this district is not now carried by electric power. The heaviest general load is in and near Phœnix, but on account of a prospective increase in the mining region eastward and northward the load center has been fixed arbitrarily at Miami. The future growth of load is problematical and depends largely on the mining industry. An estimate of the future growth of load by Col. William Kelly, Corps of Engineers, U. S. A., M. Am. Soc. C. E., Chief Engineer of the Federal Power Commission, has been published in the Second Annual Report of the Commission. This estimate is based on more detailed data concerning mining load than are available to the speaker, and, therefore, has been adopted as a whole.

Col. Kelly estimates a public utility mean load of 30 000 kw. and a mining load of 56 000 kw. in 1926. He assumes that the public utility load will increase at 9% until 1925, 8% until 1930, 7% until 1935, and 6% until 1940; also, that the mining load will increase at 5.2% until 1935, and at 4% thereafter. On these rates, and with his 1926 figures as a basis, the loads in 1921, 1930, and 1940, have been plotted on Fig. 28, as follows:

Year.	Kilowatt-hours.	Equivalent mean horse-power.
1921	539 000 000	82 400
1930	957 500 000	146 500
1940	1 610 100 000	246 300

The figures for 1921 include the actual load on existing electric systems and also the mining load.

Power Resources

The only comprehensive estimate giving the total potential power in all the streams in the Pacific drainage (as well as elsewhere in the United States) is that prepared by the U. S. Geological Survey in 1908. This general estimate has been republished several times and with certain revisions. It was made on a theoretical program with total fall and estimated stream flow as its principal basis, without reference to the engineering features of the power sites along the streams. The plant efficiency adopted in the original estimate was much higher than is attained under present practice. A revision published later* and on a more conservative plant efficiency, credits the seven States entirely within the Pacific drainage with a maximum of 31 437 000 h. p., or 58.32% of the total water power of the entire country. The figures by States are given in Table 17.

TABLE 17.—Estimates of the Minimum and Maximum Potential Water Power Resources of the Seven States Entirely Within the Pacific Drainage

State.	Total Potential Water Power.			
	Minimum.		Maximum.	
	Horse-power.	Percentage of total United States.	Horse-power.	Percentage of total United States.
Arizona	893 000	3.20	1 698 000	3.15
California	3 424 000	12.25	7 818 000	14.50
Idaho	2 362 000	8.45	5 067 000	9.40
Nevada	172 000	0.62	276 000	0.51
Oregon	3 148 000	11.27	6 613 000	12.27
Utah	743 000	2.66	1 318 000	2.45
Washington	4 932 000	17.65	8 647 000	16.04
Total	15 674 000	56.10	31 437 000	58.32

In addition to many large developments already under Government permit at the date of the Federal Power Act, the Federal Power Commission has received applications aggregating millions of horse-power throughout the country. As some of these applications have not been finally reported on (and estimates of their power capacities are, therefore, somewhat indefinite) and there are conflicts between others, it is difficult to ascertain exactly the net horse-power they represent. A rough estimate with conflicts or duplications eliminated gives for the projects in the seven States in the Pacific drainage the following approximate power, available 90% of the time:

* Table 2, Senate Doc. 316, 64th Cong., Second Session.

Arizona	3 650 000 h. p.
California	2 600 000 "
Idaho	100 000 "
Nevada	....
Oregon	700 000 h. p.
Utah	1 400 000 "
Washington	1 050 000 "
Total	9 500 000 h. p.

The most important divisions of the 800 000 sq. miles of the United States west of the Rocky Mountains, as regards water power, are:

(*a*) The Columbia River drainage, 259 000 sq. miles.

(*b*) The Colorado River drainage, 244 000 sq. miles.

(*c*) The coast drainages, comprising in order southward:

(1) The area tributary to Puget Sound, 14 300 sq. miles.

(2) The coast areas in Washington and Oregon, and in California north of San Francisco Bay, tributary directly to the Pacific Ocean, 44 700 sq. miles.

(3) The Sierra Nevada streams draining into San Francisco Bay, or into the closed basins of Tulare and Kern Lakes, the entire area in the Great Valley embracing 59 800 sq. miles; also Owens Lake Basin.

Few important power sites are to be found in the 200 000 sq. miles embraced within the arid mountain ranges and desert valleys of the Great Basin extending eastward from the base of the Sierra Nevada through Nevada and Utah to the base of the Wasatch Range, and thence southeastward through the plateau region north of the Grand Canyon. Fortunately, this region is as destitute of power demand as it is of power resources.

Few power sites are found on the coast streams emptying into the Pacific Ocean south of San Francisco Bay, and such as there are have been almost completely utilized by existing plants.

For the reasons just stated, little study need be given to the Great Basin and South Pacific drainages, and attention may be concentrated on the Columbia, the Colorado, and the Coast drainages south to San Francisco Bay.

Columbia River Drainage.—The drainage area of the Columbia above its mouth (including 34 700 sq. miles in Canada on the main stream and several of its tributaries) is 259 000 sq. miles. Of this total, 237 000 sq. miles are above The Dalles, where the stream breaks through the Cascade Range, and 32 000 sq. miles are embraced within the drainage of the main stream below The Dalles, and several tributaries, of which the most important—the Willamette and Cowlitz—drain large valleys lying between the Coast Range and the Cascades, south and north of the Columbia.*

* For a map showing the areas of various divisions of the Columbia drainage, see Fig. 29.

The Columbia River rises in Canada, flowing first nearly northward and then southwestward, and crossing the International Boundary in the extreme northeastern corner of the State of Washington. Near the mouth of Spokane River, the Columbia turns west along the base of a secondary mountain system extending eastward from the Cascades. At the mouth of the Okanogan, the stream again turns southward, and then eastward to the mouth of Snake River, where it again turns abruptly to the west, finally to break through the Cascades and discharge into the Pacific Ocean. Eastward and northward from the great loop of the river (extending from the mouth of the Spokane to the mouth of the Snake) is a series of great lava flows that cover the floor of the basin eastward to the mountains of Idaho and northward nearly to the Canadian Boundary. Much of this lava is covered, in the lower parts of the main basin and on the tributaries, with deep sedimentary soils. Clark Fork (the principal head-water tributary of the Columbia in the United States) encircles the north, and Snake River encircles the south, of the mountain ranges of Central Idaho, which are a western extension of the Rockies. Rainfall in the Columbia Basin ranges from 8 to 80 in., or more; the head-waters of the main stream and many of its principal tributaries are well forested.

The most important tributaries of the Columbia in total potential water power, taken in order down stream, are Clark Fork, Snake River, and the Deschutes. Clark Fork should be listed after Snake River if the streams are arranged in the order of magnitude of potential power. Tributaries with smaller power possibilities will be mentioned subsequently.

The Rockies on the eastern boundary of the Columbia drainage reach elevations of 10 000 ft.; elevations in the detached ranges of Idaho are rarely more than 8 000 ft.; elevations in the mountains north of the basin (along the Canadian Boundary) are rarely more than 5 000 ft., and elevations to the southward near the heads of the Owyhee, John Day, and Deschutes are from 6 000 to 7 000 ft. The highest elevations in the Columbia drainage are along its western boundary in the Cascade Range through which the Columbia has broken its way to the sea. The backbone of the Cascades in Oregon is, in general, more than 6 000 ft. in elevation, with peaks from 8 000 to 11 225 ft.; in Washington, the general elevation of the ridge is from 4 000 to 6 000 ft., or more, with peaks from 10 000 to 14 363 ft. As a result of these topographic features, the tributaries of the Columbia draining the Cascades have, in general, heavier gradients and smaller drainage areas than its eastern tributaries.

Where the Columbia crosses the International Boundary the elevation is about 1 340 ft. above sea level, and the distance from the mouth of the river is about 685 miles; at the "Cascades", the lowest power site on the main stream, the elevation is 6 ft. above sea level, and the distance is 143 miles from the mouth; the fall in the 542 miles between these two points, therefore, averages only 2.5 ft. per mile. The gradient of the stream, however, is by no means uniform, and most of the drop is concentrated at rapids or low falls, few of which are very pronounced. The great potential power of the stream is due to its high discharge rather than its rapid fall.

The power sites that have been most carefully investigated to date are, in order down stream: Kettle Falls, Priest Rapids, Grand Coulee, Umatilla Rapids, The Dalles, and the Cascades.

The site at Kettle Falls, now under permit from the Federal Power Commission to Washington Water Power Company, is unusual in that it can be developed serially in six stages that will bring the power capacity, successively, to about 13 000, 25 000, 49 000, 74 000, 98 000 and 123 000 h. p. The investment in the project, in a measure, will be proportionate to the successive steps, which should be of great advantage.

A dam has been planned near the head of Grand Coulee to furnish power for the so-called Columbia Basin pumping project (an alternative for the development of the Clark Fork storage and gravity canal) to irrigate 1 753 000 acres of land in that part of the basin bounded on the west by the Columbia and on the south by the Snake.

The important site at Priest Rapids is now under permit from the Federal Power Commission to Washington Irrigation and Development Company. The rapids at this point fall 90 ft. in 9 miles, and the dam and power plant are to be built at the foot of the rapids. The Company states that the average flow during the low-water period is 50 000 sec-ft. and during the summer months the discharge is more than 400 000 sec-ft. The dam will be 90 ft. high and about 2 miles long. Primary power at the site has been variously estimated to a maximum of 400 000 h. p., with an additional 300 000 secondary h. p. available chiefly during the irrigation season. The supporters of the project have made extensive investigations of various natural resources of the region, that may be developed through the construction of the plant.

The State Engineer of Oregon made preliminary investigation of the site at Umatilla Rapids and found that a head of 35 ft. with a minimum flow of 40 000 sec-ft. was available and estimated that 120 000 h. p. could be developed continuously, and 320 000 h. p. for the irrigation season, at a cost estimated, in 1916, at $20 500 000 based on the 320 000 h. p. capacity. This estimate, however, would have to be adjusted to present prices.

A much more detailed study of The Dalles by the State of Oregon showed an available head ranging from 45 to 105 ft., depending on the stage of the river, with a minimum flow of 50 000 sec-ft., 480 000 h. p. available continuously, 120 000 h. p. additional for 11 months, 100 000 h. p. further additional for 10 months, and 100 000 h. p. still further for 8 months, giving a total of 800 000 h. p. available for 8 months or more, at a total estimated cost of $50 000 000, with prices as in 1916.

A rather careful preliminary study of a site at the Cascades, 4 miles below Cascade Locks, was also made by the State and showed a head of 44 ft., at minimum flow of 50 000 sec-ft., with an estimated capacity of 200 000 h. p. continuous, 90 000 additional h. p. for 11 months, 120 000 h. p. additional for 10 months, and 220 000 h. p. additional for 8 months, making a total of 420 000 h. p. available for 8 months or more at a cost of $11 000 000, exclusive of the dam for which there were no trustworthy data.

Snake River.—This river, the most important tributary of the Columbia, both in area drained and in potential power possibilities, rises in the mountains north of Jackson Lake in Northwestern Wyoming, flows first in a southerly and then in a westerly direction to a junction with the Columbia River in Central Washington, 300 miles above its mouth. The drainage area of Snake River is 108 600 sq. miles. Several power sites in Idaho have already been developed, and the stream is used extensively for irrigation. Further development of storage in the head-waters for irrigation and power are contemplated, and power development in that part of the stream in Central Idaho will be largely dependent on the flow available after irrigation needs are supplied. The discharge of springs and important tributaries in Western Idaho below the largest irrigation diversions increase the supply available for power purposes, and the most important future power developments will probably be on that section of the Snake along the boundary between Oregon and Idaho and thence down to its mouth.

Profiles of the central section of the Snake River from Minidoka Dam to Enterprise, Idaho (southwest of Boise), show a total fall from Elevation 4 195 to Elevation 2 232, or 1 963 ft. in 242 miles, or a mean of 8 ft. per mile. The mean, however, has little significance as long stretches of moderate grade are followed by shorter stretches with rapid fall. The most rapid drops (certain of which are partly developed) are at Minidoka, 55 ft. in ½ mile; Milner, 200 ft. in 4 miles below the crest of Milner Dam; from the crest of Dry Creek Falls to the crest of Twin Falls, 410 ft. in 14.2 miles; from the crest of Twin Falls to the crest of Shoshone Falls, 155 ft. in 2.9 miles; from the crest of Shoshone Falls to the foot of Pilar Falls, 210 ft. in 1.7 miles; in the 2.8 miles above Rock Creek, a fall of 140 ft.; from the crest of Upper Salmon Falls to the foot of Dolman Rapids, 92 ft. in 2.2 miles; and at the Lower Salmon Falls, 35 ft. in 0.1 mile.

Lower Salmon Falls is 141 miles above Enterprise and, therefore, about 160 miles below Minidoka. The entire fall from Minidoka to Lower Salmon Falls is 1 450 ft. which leaves only a fall of 513 ft. in the 141 miles of the section of the river extending down to Enterprise. Power plants are in operation at Lower Salmon Falls, Shoshone Falls, and Minidoka, as well as at American Falls and Idaho Falls up stream from Minidoka.

Salmon River, the most important tributary of the Snake, has a comparatively regular profile on a grade broken by a series of small drops rather than long flat sections with rapid drops, as on the Snake. Profiles show that from Salmon, Idaho, to Riggins at the mouth of Little Salmon River, the stream falls from Elevation 3 935 to Elevation 1 705, or 2 230 ft. in 173 miles. The fall in the Salmon below Riggins to the junction with the Snake is 815 ft. in 86.6 miles.

Power investigations made by E. W. Kramer, Hydro-Electric Engineer, U. S. Forest Service, showed three sites with a combined capacity of more than 100 000 h. p., on the section of the Salmon between the mouths of the Middle Fork and Little Salmon.

Rather careful preliminary investigation of six power sites on a part of Snake River were made by the State Engineer of Oregon.* These projects, taken in order down stream, are Salmon River Tunnel, Mountain Sheep, Cherry Creek, Coon Hollow, Asotin, and Palouse Rapids.

The Coon Hollow and Cherry Creek sites may be considered as alternative locations to utilize nearly the same fall as that available at the Mountain Sheep site. The maximum water surface elevation above the dams, and the mininum water surface elevation for the tail-races of the three developments are, as follows: Mountain Sheep, 1 135 and 915 ft.; Coon Hollow, 1 104 and 884 ft.; Cherry Creek lies between the other two. The tail-water of the Salmon Tunnel site is Elevation 1 160.

The Salmon River Tunnel project contemplates the diversion of Salmon River through a narrow ridge to a drop on Snake River about 80 miles above Lewiston, Idaho. The net head available would range from 505 to 530 ft. The normal minimum flow would be 4 000 sec-ft. producing 200 000 continuous h. p., estimated, at 1916 costs, to total $19 215 000, or an average of $96.08 per h. p. The installation could be increased to 300 000 h. p., giving an additional 100 000 h. p. from April to October, with a total cost of $25 320 000, or $84.40 per h. p. This estimate and the following estimates, as in the case of those on the Columbia, would have to be adjusted to present-day prices.

Mountain Sheep site is 55 miles above Lewiston, Idaho, on Snake River, and 3 miles above the mouth of Salmon River. The net head would range from 200 to 220 ft., the flow from 6 000 sec-ft. minimum to 10 000 sec-ft., with the addition of Salmon River through the diversion tunnel mentioned previously, or by storage as mentioned subsequently for the Coon Hollow project. The discharge could be increased to 15 000 sec-ft. with both the Salmon River diversion and the Coon Hollow storage. It was estimated that 100 000 h. p. could be made available continuously, 200 000 h. p. with either of the alternative supplementary water supplies, and 300 000 h. p. with both the supplementary supplies. The estimated cost for the development of 200 000 h. p., exclusive of the cost of supplementary supplies, was $14 000 000, or $70 per h. p., and the cost of 300 000 h. p., on the same basis, was $16 400 000, or $54.70 per h. p.

Cherry Creek site is on the Snake River, 51 miles above Lewiston, 1 mile below the mouth of Salmon River. The head is 200 ft., the minimum discharge 10 000 sec-ft., or 15 000 sec-ft. for an average of 11 months; it was estimated that 200 000 continuous h. p. could be developed, at a cost of $13 600 000, or $68 per h. p.; and that 300 000 h. p. could be developed with storage at an estimated cost of $18 100 000, or $60.35 per h. p.

Coon Hollow site is 45 miles above Lewiston; the available head is from 200 to 220 ft. and the water supply the same as that for the Cherry Creek project. It was estimated that 200 000 continuous h. p. would cost $15 200 000, or $76 per h. p.; and that 300 000 h. p. would have been available for 11 months, or 300 000 h. p. continuous with the possible use of the existing storage.

* "Oregon's Opportunity in National Preparedness", *Bulletin 5*, Office of State Engineer, Salem, Ore., February, 1916.

The Asotin site is above Asotin, Wash., the head being 60 ft. and the minimum stream flow, 10 000 sec-ft. With storage, 15 000 sec-ft. could be obtained. On the basis of the minimum flow, 54 000 h. p. could be developed at an estimated cost of $5 400 000, or $100 per h. p., and with storage, 80 000 h. p. could be developed at an estimated cost of $6 600 000, or $82.50 per h. p.

The Palouse Rapids site is 7 miles below Riparia, Wash. The head is 32 ft., the minimum flow 12 000 sec-ft. and the flow with storage, 17 000 sec-ft. It was estimated that 50 000 h. p. could be developed, corresponding to a greater flow than the minimum, because the irrigation load comes during the high water. The estimated cost was $4 500 000, or $90 per h. p.

Clark Fork of the Columbia.—Clark Fork, the most important head-water branch of the Columbia in the United States, rises on the western slope of the Continental Divide just west of Butte, Mont., and flows in a westerly and then a northerly direction, finally emptying into the Columbia a short distance north of the Canadian boundary. The total length of the stream is 420 miles, and its drainage area at the mouth is 25 800 sq. miles. Altitudes within its basin range from 1 346 ft. above sea level at the mouth to more than 8 000 ft. at the summit of some of the highest peaks. From the boundary to the mouth of the river, the fall is about 400 ft. in 16 miles, so that where it crosses into Canada the elevation is 1 746 ft. above sea level.

The flow of Clark Fork is subject to the natural regulative effect of several large lakes that in the future may be greatly increased. These lakes are Flathead Lake (Elevation 2 881) on Flathead River, the principal tributary, Lake Pend Oreille (Elevation 2 059) on the main stream, and Priest Lake (Elevation 2 439) on the head-waters of Priest River, a tributary. There is only one important power site on Clark Fork above the mouth of Flathead River (Elevation 2 482)—the so-called "Fish Creek Site" of about 20 000 h. p. Mr. Kramer has made a preliminary study for the development of Flathead River and Clark Fork down to Lake Pend Oreille, for power purposes, and finds that, in addition to the existing developments, 621 000 h. p. is available in eight sites, with complete regulation of Flathead Lake. Between Lake Pend Oreille and the border, the most important sites are at Albany Falls, Metaline Falls, and Z Canyon. The Federal Power Commission has received application for a project at Z Canyon that will develop 195 000 h. p. The construction of a high dam at Z Canyon would cause back water to Albany Falls.

The completion of the so-called Columbia Basin Gravity Project would develop two sources of power: (*a*) the power at the canal drops on the project itself; and (*b*) the power on the Flathead and Clark Fork Rivers regulated by storage in Flathead Lake for irrigation in the Columbia Basin. The former would be seasonal power available in varying amounts from April to October of each year and reaching a maximum of 189 955 h. p. in August, with a mean of 132 889 h. p. for the seven months, of which a mean of 122 650 h. p. would be available for general sale and the remainder would be used in irrigation pumping on the project itself. The latter would give year-round power in the 820-ft. fall between Flathead Lake and Pend Oreille Lake, varying, according to past records, between a maximum of 333 800 h. p. (1910) and a

minimum of 132 970 h. p. (1920). The mean for the period, 1908 to 1920, was 257 205 h. p.; about 190 000 h. p. would have been available for 90% of the time. As shown, such a development in connection with irrigation would reduce the total output below that available with the stream developed for power alone.

Investigations made by the Commission of Conservation of Canada show an important site on Clark Fork, immediately above its junction with the Columbia, where a head of 100 ft. and a continuous output of 73 000 h. p. may be secured.

Deschutes River.—Deschutes River and one of its principal tributaries, Metolius River, draining the east side of the Cascades in Central Oregon and emptying into the Columbia near The Dalles, are very important streams in total power available and for the reason that they are comparatively near existing markets and the sites are such that development can be made in moderate sized units, following market demand. Sites on both streams have been investigated by the State Engineer of Oregon and by the Federal Power Commission. The report of the Commission shows fifteen sites in a stretch of 111 miles above the mouth of the Deschutes, with heads ranging from a minimum of 32 ft. to a maximum of 270 ft. and, in general, from approximately 70 to 140 ft. The capacities of the sites ranged from 11 300 to 83 500 h. p., but were in general from 25 000 to 50 000 h. p. The costs ranged from $92 to $205 per h. p., as estimated, for 85% load factor and delivered to the Columbia. The costs of many of the projects range from $95 to $175 per h. p.

Estimates for four sites on Metolius River showed heads of 75, 260, 400, and 300 ft., respectively, power capacities from 9 100 to 35 500 h. p., and unit costs from $116 to $252 per h. p.

Willamette River.—The most important undeveloped power sites in the Willamette Basin are on its tributaries, the Clackamas, Santiam, and McKenzie. On the Clackamas, Portland Railway, Light and Power Company has begun a progressive development and is operating two power plants. A continuous capacity of about 143 000 h. p. may be developed at six other sites for utilization in Portland and vicinity as the demand increases; the development of other tributaries will probably be completed later.

Other Tributaries of the Columbia.—The estimates just given in the Columbia drainage are based on rather careful investigations of specific sites. Theoretical computations of horse-power available on other tributaries* are as follows: Yakima River and tributaries, 34 000 h. p.; Natches River, 57 530 h. p; Klickitat River, 154 000 h. p.; White Salmon River, 105 000 h. p.; Little White Salmon River, 8 550 h. p.; Lewis River, 78 000 h. p.; Toutle River, 44 000 h. p.; and Cowlitz River, 175 000 h. p. On account of topographic or other difficulties, it will probably be found that not all the capacity of these streams can be developed.

Coast Drainages

Puget Sound.—The streams running from the Cascades westward into the east side of Puget Sound and from the Olympic Mountains eastward

* "Water Powers of the Cascade Range", U. S. Geological Survey, *Water Supply Papers Nos. 253, 313,* and *369.*

into the Sound and northward into the Strait of Juan de Fuca, afford many opportunities for economical power development in units of moderate size. The drainage basins of these streams are comparatively small, the largest being that of the Skagit River with about 2 500 sq. miles at a point below all important power sites; the total drainage area tributary to the Sound is only 14 700 sq. miles. The elevations in the Sound drainages range from sea level to a maximum of 14 363 ft. at the crest of Mount Ranier. The rainfall ranges from 40 or 50 in. along the shore of the Sound to a maximum of about 150 in. near the crest of the Olympic Mountains. The geological structure is volcanic. Lakes are numerous, and the stream flow is comparatively abundant and regular. The gradients of the streams are far steeper than in large rivers like the Snake and Columbia. Most of the existing and proposed developments are of the low or moderate head type.

The City of Tacoma is about to start active construction of a large development on the Skokomish, and its system should have an ultimate flat load capacity of 80 000 h. p.

On the north end of the Olympic Peninsula, applications have been filed for several sites on the Soleduc and Elwha Rivers. These developments should have a flat load capacity of 70 000 h. p., or more.

Skagit River has claimed much attention since the City of Seattle determined on its utilization for municipal power purposes. An excellent storage site at the head of a series of three projects will permit the development of 158 000 continuous h. p., and 10 000 h. p. additional is available on the Cascade River, a tributary in the vicinity of the system on the main stream. Additional important sites are available on other tributaries.

Other streams flowing into the east side of the Sound are important sources of power in various stages of development. Theoretical computations* give the following power capacities: Nisqually River, 35 700 h. p.; Puyallup River, 40 400 h. p.; White River, 102 000 h. p.; and Cedar River, 37 870 h. p.

North Pacific.—The coast streams in Washington, and in Oregon and California from the Columbia southward to San Francisco Bay, drain an area of about 44 700 sq. miles lying entirely on the west side of the Olympic, Coast, and Cascade Ranges, with the exception of the head-waters of Klamath River which drain some territory in, and to the east of, the Cascades. The largest single drainage basin is that of the Klamath, which has an area of about 16 000 sq. miles above the mouth. The next streams in size and importance are Rogue River and Umpqua River. Elevations within the Klamath Basin vary from sea level to about 9 000 or 10 000 ft. at the summit of several peaks in the head-waters basin and a maximum of more than 14 000 ft. on Mt. Shasta; elevations in the Rogue River basin range up to 10 000 ft. The highest point in the Umpqua Basin is about 8 000 ft. All the basins are, in general, well forested and have mean annual precipitations of from 40 to 100 in., or more, except the inland parts of the Klamath Basin, which are cut off from the ocean by intervening ridges that reduce the rainfall to 10 in., or even less, in the extreme eastern part of the drainage.

* U. S. Geological Survey, *Water Supply Paper No. 313.*

Although both Klamath and Rogue Rivers are far from the important load centers of Central California and the Portland District in Oregon, plants of The California Oregon Power Company on both streams are now transmitting power north and south to these markets, in addition to serving local demands. The future use of the Klamath River may be considerably affected by irrigation development.

Umpqua River, draining into the ocean in Central Oregon, although an important power stream, is as yet practically untouched. Careful estimates by Mr. Kramer show that with head-waters storage, a continuous output of 220 000 h. p. may be obtained at fifteen different sites, seven of which, with a combined capacity of 161 000 h. p., are known to be very desirable. The sites are of low or medium head.

On Rogue River, two sites controlled by The California Oregon Power Company were reported by the State Engineer of Oregon to have heads of about 100 ft., and continuous capacities of 12 000 and 13 000 h. p. The unit costs were $70 and $130, respectively, at prices prevailing in 1916. These two sites and the partly developed sites do not comprise the total power on the stream.

Klamath River affords many opportunities for the development of power in the 255 miles between Klamath Lake in the head-waters basin and the ocean. The total fall in this distance is about 4 100 ft., and the gradient of the stream is comparatively regular, most of the drop being in rapids rather than falls. Two low-head plants are already in operation below the outlet of Klamath Lakes Basin.

There are many dam sites throughout the canyon and the height to which future dams are built, or the number of projects to be developed, will depend largely on market conditions. Of the power capacities plotted on Fig. 28, the square at the mouth of the Salmon represents 67 500 h. p. in three projects; and the square above the mouth of Shasta River represents three proposed developments and one site with a combined capacity of 42 000 h. p. There are other opportunities on the middle river for the development of at least 70 000 h. p.

Central California Streams.—The Great Valley of California is drained by the Sacramento and San Joaquin Rivers (the San Joaquin joining the Sacramento just above the point where the latter empties into San Francisco Bay), and by several streams discharging into Tulare and Kern Lake Basins. The total area above the outlet into the Bay (including the Kern and Tulare Lake Basins at the south end of the San Joaquin Valley, but not naturally tributary to this river) is 59 500 sq. miles. In this area only the streams draining the western slope of the Sierra Nevada, and the extreme southern end of the Cascade Range, have important power possibilities. The most important of these in order southward are Pit River, Feather River, Yuba River, and American River, all tributaries of the Sacramento, and Mokelumne River, Tuolumne River, Stanislaus River, Merced River, San Joaquin River, Kings River, and Kern River, all tributaries of the San Joaquin, or draining into the Tulare and Kern Basins. Power sites of minor importance are found

on the head-waters of the Sacramento River, Deer Creek, Mill Creek, Kaweah River, and Tule River.

The power possibilities of nearly all the streams mentioned have been at least partly developed. The elevation of the canyon mouths of most of the streams is between 500 and 750 ft. above sea level and the elevation of the head-waters becomes progressively greater to the south. The only one of these streams that drains the territory east of the Cordillera is Pit River, which heads in the interior basin in the Warner Mountains in Northeastern California and breaks through the southern end of the Cascades between Mount Shasta and Lassen Peak. Feather River drains a large mountain valley that can hardly be classed, however, as part of the interior basin. The Feather River drainage, although usually assumed to be wholly in the Sierra Nevada, is really the dividing line between the Cascades and the Sierra. The difference in structure between the two ranges is well marked in the northern and southern parts of the basin, Lassen Peak on the northern boundary being a typical volcanic cone of the Cascades and the peaks of the southern part of the basin are granite, the distinguishing formation of the backbone of the Sierra Nevada. This granite backbone extends south through the entire length of the range to the spurs south of Mount Whitney, and is flanked in the foothill belt to the west by andesites and shales; at only a few points, as in the Mono Lake region, is there any volcanic formation such as is typical of the Cascades.

The stream flow characteristics change toward the south, being affected not only by geology, but by topography and by changes in rainfall. Pit River and the North Fork of Feather River are spring-fed streams of the general character found in the Cascades. The Middle Fork of Feather River is more nearly typical of the streams of the Sierra in its flow than those of the Cascades, and the general tendency to the south is toward well marked low-water and flood seasons. The yield per square mile also has a general tendency to decrease toward the south, particularly from Kings River to the end of the range. The heaviest rainfall is in the head-waters of the North Fork of Yuba River, where the annual mean is about 80 in. Toward the south, the rate of rainfall decreases, first rather rapidly and then more slowly to a mean of not more than 15 or 20 in. in the head-waters of the South Fork of the Kern and 10 in. or less in the foothills of the Kern.

The flow of streams that head on the main ridge of the Sierra is sustained by melting snows at the higher elevations, but those streams that do not penetrate to the higher regions (or touch the main ridge only at one or two points, such as the Merced) must depend to a greater extent on storage. The most flashy of the Sierran streams are probably those heading on the west face of some of the secondary ridges to the west of the main ridge. Tule River, heading on the west side of the Great Western Divide, is an example of this class. The rapid gradient, the large areas of exposed granite, and the relatively small size of its drainage area combine to give it a particularly rapid run-off, with only little high country to maintain the low flow from melting snow.

In few sections of this country are the benefits of water storage so well exemplified as in some of the secondary drainages of the Southern Sierra. Big Creek of the San Joaquin drainage may be cited as an example where construction of storage has made possible a series of plants now partly completed. Before the storage was constructed the minimum flow of Big Creek was less than 3 sec-ft., corresponding to about 1 000 h. p. with 4 000-ft. fall. With storage, two plants with a combined head of nearly 4 000 ft. out of a total of 5 000 ft. ultimately available, and reservoirs only partly developed have an average annual output of nearly 40 000 h. p.

Pit River and its principal tributary, the McCloud, offer excellent opportunity for the development of power. Both streams are spring-fed and power plants may be operated successfully without storage. Most of the sites are of the low-head type, but the largest has a head of about 800 ft. Within the past two years, two plants have been built on Hat Creek and one on the main Pit by Mount Shasta Power Corporation, a subsidiary of Pacific Gas and Electric Company, as the first steps in a series of seven plants, five of which are to be on the main river. Two additional projects are under permit from the Federal Power Commission to the Pit River Power Company and three sites on the Pit and McCloud near the junction of the two streams have been filed on or are under permit to individual applicants. The total continuous power of the undeveloped sites is more than 465 000 h. p.

Feather River shares with the Pit a pre-eminent value for the development of water power. Two plants and a large reservoir on the North Fork, a spring-fed stream, form the first steps in a serial development of that branch by Great Western Power Company of California. As planned, this series will consist of six plants. Surveys are already under way for the construction of a combined power and irrigation storage and a series of power plants on the Middle Fork of the Feather River. It is uncertain, however, what the final arrangement of this system will be. In addition to the development of the main branches of Feather River, there are attractive opportunities for the development of their principal tributaries, and 500 000 continuous h. p. is a conservative estimate of the undeveloped energy in the basin.

Of the three branches of Yuba River, North, Middle, and South Forks, the South Fork is utilized largely by the existing developments of Pacific Gas and Electric Company, which company also owns one plant diverting from North Fork. Undeveloped sites on North and Middle Forks have been applied for by two power companies and an irrigation district. When the present conflicts between the various applicants have been disposed of, and an allowance has been made for sites on South Fork, the total capacity of undeveloped sites in the basin will be about 265 000 h. p.

The canyon of the North Fork of American River is cut too deeply to afford attractive power sites without storage. From time to time, the development of Middle Fork has been proposed by various companies, but at present this project seems to be dormant. On South Fork and its tributaries, operating companies and individuals have filed on three projects with a combined

capacity of about 80 000 h. p. and on one of the developments construction is under way. Minor developments have been in operation on this stream for many years.

Mokelumne River has been utilized for many years by one plant, and above this plant and below the reservoirs used in its operation, are undeveloped sites. The three plants proposed by present applicants would have a continuous combined capacity of 50 500 h. p.

Tuolumne River is widely known as the source of the Hetch Hetchy power and water supply system of the City of San Francisco. Only three very small water-power developments have hitherto been in operation on the stream or its tributaries. In the near future, the city will complete its Moccasin Creek plant with a continuous capacity of 70 000 h. p. and, in subsequent years, will develop a plant between Hetch Hetchy Reservoir and Early Intake of the aqueduct and another plant utilizing the water supply diverted from Cherry Creek and Eleanor Creek to a permanent plant on the north bank of the river near Early Intake. A power company has applied to the Federal and State authorities for permission to develop storage and power on the South Fork of the Tuolumne. The combined continuous capacity of all these plants, when completed, will be 170 000 h. p., in addition to which Turlock and Modesto Irrigation Districts will develop considerable seasonal power at their Don Pedro Irrigation Dam on the lower river. The power here available has not been included on Fig. 28.

San Joaquin River is the most important power stream in the south central part of the Sierra Nevada. The lowest power development will probably be at a proposed irrigation dam at about Elevation 450; the tail water of the only existing plant on the main river is at Elevation 650 and the total drainage area above the lowest power plant will be 1 640 sq. miles. Elevations range from 11 000 ft. to more than 13 000 ft. along the crest of the Sierra in the head-waters. Much of the basin is well forested. With one exception, the developments to date have been on secondary streams such as the North Fork of the San Joaquin, which is occupied by a series of plants of San Joaquin Light and Power Corporation, and Big Creek, which is utilized by another series belonging to Southern California Edison Company. The latter company proposes large extensions of its present system both on the main stream and on the principal tributaries. Although the locations now proposed are not final, it appears that projects of the series, as yet undeveloped, will furnish a continuous output of 500 000 h. p.

Kings River, second only to the San Joaquin for potential power in the southern part of the Sierra, is as yet undeveloped. As on the San Joaquin, the lowest power development will probably be at an irrigation dam where the stream leaves the foot-hills. The drainage area above that point is about 1 700 sq. miles, and elevations range up to about those found in the head-waters of the San Joaquin. The drainage area is much more rugged and good forest cover is found chiefly in the secondary basins such as that of the North Fork. This branch of the river is of the same character as Big Creek of the San Joaquin, the stream dropping at a tremendous gradient

from a plateau basin to the main river and offering, as on Big Creek, feasible heads of 2 000 ft. The series of plants will depend largely on storage. A complete plan of development for the North Fork has been made by San Joaquin Light and Power Corporation. On the two principal branches of the stream, Middle and South Forks, the City of Los Angeles has made preliminary plans for a series of developments; minor branches of these two streams would probably furnish additional power. A conservative estimate for all contemplated developments on Kings River shows a continuous output of not less than 400 000 h. p., providing certain upper storage sites prove to be feasible.

Kaweah River, south of the Kings, drains the western slope of secondary ridges of the Sierra, cut off almost entirely from the main Divide by the head-waters of South Fork of Kings and the North Fork of Kern River. The stream, therefore, does not receive so great a benefit from melting snows and natural storage in small lakes. Much of the Middle Fork area is well forested. The few attractive power sites are developed at present almost entirely by existing systems of Southern California Edison Company.

On Tule River, the only potential powers still undeveloped are extensions of an existing plant owned by San Joaquin Light and Power Corporation. The stream, therefore, is of little future potential value.

Kern River, the southernmost of the important streams draining from the Sierra Nevada, has a total area of more than 2 000 sq. miles above the mouth of the canyon, ranging in elevation from 850 to 14 500 ft. Much of the drainage area is dry mountain country lying in the foot-hills, or in the comparatively arid ridges in the South Fork Basin. The most important power sites are on North Fork and on the main river below the junction of North and South Forks. Southern California Edison Company has under contemplation and construction a complete system for the utilization of North Fork and the main river. Three of its plants and one belonging to San Joaquin Light and Power Corporation utilize a considerable part of the total fall. The continuous power capacity of the remaining sites of the series and two projects applied for by the City of Los Angeles on South Fork amount to approximately 62 500 h. p. Future development of the stream will probably be delayed pending the completion of more attractive developments on other drainages, and the final program may be affected by the proposed construction of a large storage reservoir, primarily for use in irrigation, at the junction of the South and Middle Forks.

Owens River drains part of the eastern slope of the Sierra. The development of the principal sites on Owens River and its tributaries has been planned by the City of Los Angeles in connection with its aqueduct and municipal water and power system. The most important sites are at Owens River Gorge below the proposed Long Valley storage reservoir of 340 000 acre-ft. capacity. The total fall in the gorge, not all of which is controlled by the city, is 2 300 ft. With complete regulation, about 56 000 h. p. could be developed in a series of plants utilizing the entire fall. Other developments

on minor streams (of small drainage area, but very rapid gradients) would bring the total continuous output for the Owens Valley region to 80 000 h. p.

On the east side of the Sierra, there are only a few streams, aside from Owens River, that are of great power importance. Truckee River is already well developed, Leevining and Rush Creeks emptying into Mono Lake are in process of development and proposals have been made to develop the minor sites on Carson and Walker Rivers.

Colorado River Basin.—Few streams have received so much recent publicity as the Colorado River, which drains parts of Colorado, Wyoming, Utah, Arizona, New Mexico, Nevada, and California. The stream is extremely valuable as a source of supply for both irrigation and power. It is naturally divided into three sections: The head-waters basin, above the mouth of the San Juan; the canyon region, from that point down to the mouth of Virgin River where the Colorado, after flowing in a general southwesterly direction, turns almost due south; and the valley region, from the mouth of Black Canyon southward to the Gulf of California. The entire drainage area above Yuma is 244 000 sq. miles. Of the entire flow at Laguna Dam which is above the mouth of the Gila (the principal tributary in Arizona), more than 90% originates above the Grand Canyon. The most important potential powers are between the mouth of the San Juan and the mouth of Black Canyon, the total fall between the two points being about 2 500 ft. There are, however, large water powers in the upper basin on the two branches, particularly on Green River.

Estimates of the proposed capacity of the various power projects on the Colorado River are greatly complicated by conflicts and by different programs for river regulation. At certain points as many as three projects are proposed for the utilization of a given fall. In showing, on Fig. 28, the power available at some of the most important sites, the capacities in the upper basin are those reported by the Federal Power Commission; the capacities from the mouth of the Green to Diamond Creek, inclusive, are on an assumed ultimate regulation of 10 000 sec-ft. through the approximate heads, and the power capacities from Lower Boulder Canyon to the Parker site, inclusive, are on the assumed final power regulation to 15 000 sec-ft. through the available heads.

These assumptions should be kept in mind in comparing the power of these sites with that made available by any other program of regulation or development.

The Colorado River Problem and the California Power Supply.—The Colorado River problem has a marked bearing on the power situation throughout California.

Early in their history the operating companies in Southern California had to go far north into the Southern Sierra Nevada for their hydro-electric supply; similar conditions, although not so acute, also caused the companies of Central California to develop plants north of their market. The general trend of transmission in the State is, therefore, from north to south. This condition may be changed by the building of a single plant on the

Colorado, by the Federal Government, municipality, or public utility. A large block of power will be made available in a single step, and the utilities and mines of Arizona will be able to absorb only a small part of this power. The utilities and municipal systems of Southern California will be the only other prospective purchasers. The companies and the City of Los Angeles have already acquired rights on the streams of the Sierra and have outlined programs that are in various stages of development. Some programs, in their initial stages, are complicated by conflicts with established rights, and promise high unit cost; these developments may well be dropped with little loss, and with no thought of taking them up in the future. Others are well toward completion (at least, in part) and must be placed in operation at the earliest possible date in order to avoid enormous financial loss; once in operation their output must be absorbed into the market. Others, as yet only in initial stages, are to be completed under State or Federal grants that will lapse unless the systems are placed in operation within a limited time. With the lapse of such grants adverse titles (particularly for the use of water) will become established, and will further complicate the plans for power development if resumed at a later date. In short, a hydro-electric system operating under conditions that now exist in California must plan for years in advance not only its construction program but also its campaign to secure and maintain rights for the use of water and public lands; it is not in the relatively simple position of the steam-generating utility, that has only to add a new unit to its plant when the load increases. It appears, therefore, that even if Colorado River power is made available in Southern California at a lower switchboard cost than that immediately available from the Sierra, the problems just mentioned will affect the rate at which it is absorbed by the existing systems. Most of the questions may be solved on some practical basis, and doubtless their solution will be accelerated by the recognized benefit that will result from the final control of the Lower Colorado River.

It is to be anticipated that on the introduction of Colorado River power in the southern part of the State the amount of power transmitted southward from the Sierra will be greatly reduced and that some of the systems now transmitting both north and south from the San Joaquin drainage will send all their power to the north; the flow of power southward will certainly be reduced, and, under certain conditions, may be completely reversed.

General Summary

Granting that the previously discussed estimates of increase in power demand of various markets are approximately correct, it is certain that within the next 20 years there will be no shortage of power in the Pacific States. The only territory in which the power demands will exceed the local power resources is in Southern California and there the local shortage can be remedied by power transmitted from the central part of the State, unless in the meantime part of the great potential power of the Colorado is developed.

Colorado River power will not be absorbed into the California market merely because of a slightly lower market price. The price will have to be reduced sufficiently to justify the operating systems in abandoning exten-

sive development programs now under way. Until such time as Colorado River power is made available in Southern California, the trend of power transmission in that State will be (as in the past) toward the south, but, thereafter, the resources of the Southern Sierra may be utilized in markets lying to the northward.

Early construction at the major sites on the Columbia River itself is dependent chiefly on the development of new industries, as the initial block of power that would be brought in by any of these projects (except that at Kettle Falls), is greater than would be warranted by ordinary growth of the present market; all ordinary demands in the Columbia drainage can be met through the construction of projects on its tributaries (such as those shown on Fig. 28, or at smaller sites too numerous to be covered in this paper).

Although there is ample potential power throughout the West to supply the needs for many years, complications between power and irrigation are constantly increasing, and close study of the various problems on the most important streams should be undertaken at the earliest possible date if the best plans for their ultimate development are to be secured.

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DISCUSSION

J. M. Howells,* M. Am. Soc. C. E.—In reference to hydro-electric development in California, the speaker wishes to point out certain fortuitous geological changes of a striking character in the northeastern part of the State. The greater part of such changes took place just prior to the Age of Man and have played an important part in arranging for the recent great developments in that locality. The study by Dr. J. S. Diller and that recently made by Professor W. O. Crosby at the instance of John R. Freeman, Past-President, Am. Soc. C. E., have furnished the geological data for a better understanding of the happenings of such great import in connection with the present work.

The Pit River development of the Pacific Gas and Electric Company and the North Fork development of the Feather River of the Great Western Power Company occupy this territory. Had the topography remained as it was in the beginning of the Tertiary period, the great underground water storage of the Upper and Central Pit River Basin and the great surface water storage site at the head-waters of the North Fork of the Feather River would not have existed to attract the pioneers in hydro-electric development. About a mile south of the Big Meadows Dam, now forming Lake Almanor, a water-shed marked, in those days, the head-waters of the North Fork. North of this, instead of the present high mountains, the land lay in the form of low gently rolling hills that led the drainage north and northwesterly to the Sacramento River. The surface or country rocks of this section were mostly sedimentary.

Epoch-making events which marked the late Tertiary period moved the boundary of this water-shed north until more than 500 sq. miles of additional area became tributary to the North Fork of the Feather River and to the site of the Lake Almanor Dam. Great eruptions in the Lassen Peak District caused this change, and imperfectly fluid lava built up high mountains there, thus forming the high Lassen Peak Ridge and adding its eastern and southern slopes to the Feather River drainage. Thus, not only was additional area furnished, but also heavy precipitation and run-off were assured by virtue of the resulting lofty elevations. The great reservoir site of Big Meadows, however, was not formed until extensive and repeated floods of fluid basaltic lava, emerging from many vents, had covered not only a large part of the 500 sq. miles of the new drainage basin, but also 200 000 to 300 000 sq. miles farther north. Subsequent faulting depressed the Big Meadows section below the general plain and formed a natural lake of, perhaps, 25 000 acres, which was furnished with an impervious lining, by erosion from the surrounding country. Erosion continued until a notch was cut in the basaltic dam of the lake. This notch has been restored recently by comparatively feeble human effort, and Lake Almanor has resulted.

From this reservoir site, the Sierra Nevada Mountains extend to the southeast and the Cascade Mountains to the northwest, Lassen Peak being the southernmost of the latter and Lake Almanor forming the dividing line, separating the two great mountain ranges. The Cascades, or water-falls them-

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selves, from which the range takes its name, were caused by the hard basalt which generally lines the beds of the streams of this district and refuses to yield readily to erosion, thus causing their flow to fall in cascades.

To the north and northeast of Lassen Peak lay the basin of the Pit River, its northern boundary being just south of the Oregon line and reaching from Mt. Shasta on the west almost to the Nevada line on the east. This drainage area is in large part a basaltic plain and, where sealed by the deposits of erosion, furnishes reservoir sites. Many of these sites, however, are so large as to be out of proportion to their water supply, and, therefore, are more useful for agriculture than for water storage. Reservoirs of another form are abundant, namely, the great natural underground reservoirs which equalize the flows of such tributaries of the Pit River as Hat Creek and Fall River. Over great areas and along these streams, basaltic lava lies as if it had cooled only yesterday. It is shattered and barren and stores all precipitation as fast as it falls and conveys other streams as well. Entrapped water sinks to some impervious stratum and finally issues in great springs of remarkably even flow. The importance of this for hydro-electric purposes may be better understood when one realizes that at the point where Hat Creek enters the Pit River, it has a mean flow of about 650 sec-ft., that Fall River has a flow of 1 400 sec-ft., and that the maxima and minima flows of these streams differ by less than 15% from the mean flows. With such a great and evenly delivered water supply and a fall, in about 60 miles of river, of more than 2 000 ft., one can understand what inspired this development by the Pacific Gas and Electric Company.

The Pit River plants will include three additional developments on the main stream, all possessing a greater minimum flow than the combined flow of Hat Creek and Fall River, but as these developments take their water from the main stream below those tributaries, they also partake of the unregulated flood flow of the greater Pit River basin which is not so fortunate in underground storage, but which possesses opportunities for surface storage.

The drainage area to the south, which the Tertiary period made tributary to the North Fork of the Feather River, also possesses ground-water storage, so that the normal low flow from the new 500 sq. miles of area is at least 40% of the entire river at Oroville, Calif., where 3 600 sq. miles of drainage area reaches the plane of the Sacramento Valley. In addition to this large flow of ground-water, a large flow of surface water is yielded by the new basin. Both supplies now feed Lake Almanor, the main storage reservoir of the Great Western Power Company. This reservoir has a present capacity of 300 000 acre-ft. and, by a 40-ft. addition to the dam, a contemplated capacity of 1 250 000 acre-ft. It has an ultimate proposed spillway level of 4 500 ft. above the sea and is expected to serve eight successive power sites, the tail-race of the lowest being only 160 ft. above sea level. Thus, there is a total gross drop between lake surface and final tail-race of more than 4 300 ft.

This new drainage basin with its auxiliary—Butt Creek—when both are regulated by the ultimate dam, will yield a constant flow of 1 195 sec-ft. When used in conjunction with the greater unregulated basins which are

tributary to the five lower river sites, this regulated water supply finds its highest value.

Of the eight plants proposed, Plant No. 7 is nearly completed. Although occupying only about one-tenth of the total drop, this plant, since the commencement of operation in 1908, has led in size any hydro-electric plant west of the Mississippi River. Pit River Plant No. 1 which has recently been completed, however, by the Pacific Gas and Electric Company, exceeds by nearly 10% the size of Plant No. 7. At Site No. 2, the Great Western Power Company has begun an ultimate installation of 135 000 kw., 40 000 kw. of which is in operation.

M. M. O'SHAUGHNESSY,* M. AM. SOC. C. E.—The first power plant in Northern California was built more than 32 years ago by the Sacramento Gas and Electric Company, under the management of Mr. H. B. Livermore. A canal built by prisoners from Folsom Prison, diverted water from the American River so that a head of 55 ft. was obtained. The transmission plant was completed from Folsom to Sacramento, a distance of 22 miles, and power was delivered on September 1, 1895.

In the same year, the second plant, consisting of two 375-kw. generators wound for 5 500 volts, was undertaken on the South Yuba River about 5 or 6 miles above Nevada City, to supply power to the mines at Grass Valley and Nevada City. Since that time, the power industry has received a marked impetus, so that, in 1921, the four largest companies furnished power, as follows:

	Kilowatt-hour sales.	Revenue.
Pacific Gas and Electric Company.....	1 021 820 689	$22 898 046
Great Western Power Company.....	340 060 361	5 947 710
San Joaquin Light and Power Corporation.	313 791 835	5 084 780
Southern California Edison Co.....	840 081 210	15 074 480
Total	2 515 754 095	$49 005 016

This power is retailed to consumers at an average rate of 1.95 cents per kw-hr., or, deducting the amount paid in taxes, 1.75 cents per kw-hr.

The total hydro-electric horse-power in operation by the two municipalities of Los Angeles and San Francisco, at present (1922), is 88 450 and by thirteen private companies, 911 465, making a total of 999 915.

There are under construction, at present (1922), an additional 142 660 h. p. by municipalities and irrigation districts and 228 150 h. p. by private companies, or a total of 370 810 h. p., all of which will be installed within 2 years, and, at that time, will make a grand total for the State of 1 370 725 h. p.

The possible developments of power companies, which are financially feasible within the next 20 years, include about 4 000 000 additional h. p. These developments will not exhaust all the possibilities of the State, which may be closely estimated at from 8 000 000 to 9 000 000 h. p.

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The uses of the water of the State may be considered in the following order:

1.—Domestic use for the inhabitants.

2.—Irrigation uses for farm lands and orchards.

3.—Use of water for hydro-electric power purposes.

In some fortunate instances, the combination of all three uses may be obtained from the same drainage area.

The physical topography of the high mountains, with the limited reservoir basins and steep gradients of the river beds, makes the problem of selection of effective reservoirs difficult. The few natural lakes in the Sierra Nevada Mountains suggest the most feasible sites. They possess the advantage of freedom from silting which is incidental to foot-hill reservoirs. The climatic and topographical features of the State regulate the run-off of the streams quite differently from any of those in the Mississippi or Eastern Coast drainage areas.

Tuolumne River has an average annual flow of 2 040 000 acre-ft. from a drainage area of 1 548 sq. miles. This drainage area extends from an elevation of 300 ft. easterly, for a distance of 80 miles, to an elevation of 10 000 ft. On the higher level, above 4 000 ft., there are numerous natural lakes hemmed in by granite. The floods in the lower reaches to an altitude of 3 000 ft. are developed by winter and spring rains which occur from November to March. The second series of floods occur in May and June, and are caused by the melting snow, which may be as much as 15 to 20 ft. deep on the 6 000-ft. level. The 5 000-ft. contour is the approximate zone of maximum precipitation which diminishes at the crest of the Sierra Nevada Mountains.

The first development in this basin was made by the Turlock and Modesto Irrigation Districts. A concrete dam 127 ft. high was constructed at La Grange to elevate the water in the river bed and divert it by two canals to the irrigated areas of 165 652 acres inside the Districts, extending from Elevation 300 to Elevation 40, on the San Joaquin River. Heretofore, only two reservoirs with a storage capacity of 76 000 acre-ft. were in use along the canals to balance the flow of the river. A new dam at Don Pedro is now being constructed to create a reservoir with a capacity of more than 250 000 acre-ft. to supplement the supply.

Since 1914, the City of San Francisco has been actively constructing storage works in the higher levels, including the Hetch Hetchy Dam, with an initial storage of 206 000 acre-ft and ultimate storage of 348 000 acre-ft., and Lake Eleanor Reservoir, with an initial capacity of 28 000 acre-ft. and ultimate capacity of 168 400 acre-ft. Even with those projected works completed, there will be only about 800 000 acre-ft. in storage for a river that fluctuates annually between a minimum of 895 000 acre-ft. and a maximum of 3 700 000 acre-ft.

A consistent effort, however, has been made to develop storage on this drainage area, in favorable contrast to the Merced River, immediately south, which passes through the Yosemite National Park, with an average annual run-off of 1 228 000 acre-ft., and has storage for only one-third of 1% of its average flow, and the Stanislaus River, immediately north, with an average run-off of 1 380 000 acre-ft. and only about 10% of this volume in storage.

Only three of perhaps forty rivers in California have been mentioned that are capable of economic development. This situation discloses the great opportunity for the hydraulic engineer, the financier, the philanthropist, and perhaps the State, to undertake the conservation of waters so that the greatest good can be accomplished to the 10 000 000 acres of land capable of irrigation in California, of which only 6 000 000 acres have been brought under irrigation. A solution of these problems is being sought by the so-called Water and Power Act to bond the State for $500 000 000 for this purpose. There are fundamental objections to this Act, which make it a dangerous measure to adopt, but the speaker has hopes that from the discussion of it, a sane measure will be evolved, which will do justice to the future of California and increase the prosperity which is now being developed from its civic, manufacturing, and agricultural activities.

J. B. LIPPINCOTT,* M. AM. SOC. C. E.—The Colorado River is the great undeveloped water resource of the Southwest, and its greatest importance is the irrigation of the 2 000 000 acres of arid land below the Grand Canyon. The basic feature governing the liberation of the water of the Colorado from these reservoirs should be for the benefit, primarily, of irrigation. Second in importance is the prevention of flood overflow of the ribbon-like valleys below the Grand Canyon, and the delta, including about 800 000 acres in the Imperial Valley. There is also an equally great area of delta land on the Colorado River in Mexico, that is being rapidly developed, and protection of these lands from flood destruction is essential.

The entire late summer flow of the Colorado River near Yuma, Ariz., is now diverted, and the time has come when it is desirable to hold flood waters for periods of drought. It is hoped that the engineers of the U. S. Reclamation Service will be able to proceed with their policies of building dams along the Lower Colorado River for the impounding of flood waters for irrigation, land protection, and power development. Private capital is willing and ready to undertake these necessary enterprises in case Congress does not see fit to make the appropriations for their construction. These reservoirs are essential to the reclamation of the deserts and to the protection, both from flood and famine, of the 50 000 people who are already settled there.

If Congress does not soon act favorably on the recommendations of the engineers of the U. S. Reclamation Service pertaining to the Colorado River, as outlined and referred to by Mr. Davis, and under such Federal regulations as those described by Mr. Merrill, the Government should authorize others to proceed with these great reclamation works under a plan for construction and operation, supervised and approved by the officers of the Federal Government.

C. S. JARVIS,† M. AM. SOC. C. E.—After one of the meetings of the Colorado River Commission, during which one of the members held views divergent from the majority, some one spoke of the proponent of the minority opinion as "the stubbornest man in seven States". He had held out for unrestricted

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development of irrigation projects near the head-waters of the stream, whereas the prevailing view favored the priority of filings as a basis of rights.

The basis of the minority opinion is found in the following claims:

First.—Reduction of the flood crest, with resulting benefits to the lower projects now protected by levees.

Second.—Reduction of both the total sediment load and the percentage of transported material.

Third.—Increase of humidity in the drainage basin, with a consequent increase of precipitation and reduction of evaporation losses along the course of the stream.

Fourth.—Return flow representing a considerable proportion of the diverted waters.

In several instances, where investigations have been possible, the return flow in the Colorado Basin and adjacent territory has amounted to 30, or even 55 per cent. Where the lesser percentage applies, it seems apparent that an irrigator who diverts 1 000 acre-ft. from the flood crest, or surplus flow, effectively stores in underground reservoirs 300 acre-ft., part of which will contribute to the river during its low stages, and thus directly benefit the lower projects.

The lowest flow of the Colorado River has been completely appropriated, so that future development of the water resources in this Basin must be supplemented by storage. What is more effective storage than the underground reservoir under each irrigated tract in the uplands of this Basin? Underground reservoirs retain the water without the usual evaporation losses to which open reservoirs are subjected, and if their supply should be interrupted, they will continue to contribute for months, or even years, to the continuous flow. It is not unusual to note places in the Colorado River Basin, where a series of dry dams, maintained in a creek or river channel, successively divert the water supply for as many projects; and in extremely dry seasons, the lowest diversion is probably the most dependable supply, owing to the return flow from underground reservoirs.

The projects on the head-waters of the Colorado River, naturally, will develop more slowly than those in the lower valleys, owing to the difference in climatic and soil conditions, the cost of diversion from the canyons and gorges, and the inaccessibility of the mountain districts. Would any one, therefore, retard the natural rate of development of these projects by denying them a right to the primary flow of the stream, when their diversion of flood waters both protects and supplies the lower projects?

Of course, the speaker's judgment is that the minority view is worthy of most serious consideration, and that the most logical development of production in minerals, in live stock, in agricultural products, in storage water for the lower projects, and the best growth of markets for power and commercial products, will follow an unrestricted irrigation development in the uplands of the Colorado River Basin.

D. C. HENNY,* M. AM. SOC. C. E.—The Columbia River offers large opportunities for the development of power, which, as far as the main stream is concerned, are as yet practically untouched.

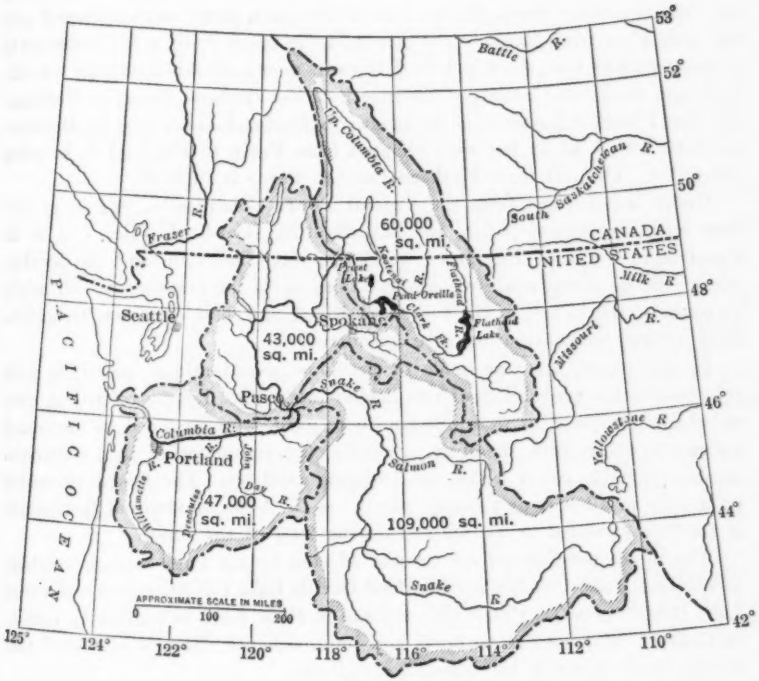


FIG. 29.

The drainage area at its mouth (Fig. 29) is 259 000 sq. miles, and its mean flow is 210 000 sec-ft. Table 18 permits comparison with a few other well known rivers.

TABLE 19.

Stream.	Point of measurement.	Drainage area, in square miles.	Mean flow, in second-feet.	Mean flow, in second-feet, per square mile of drainage area.
Colorado River	Mouth	242 000	24 000	0.10
Yukon River	Eagle, Alaska	254 000	73 000	0.35
Missouri River	St. Charles, Mo.	527 000	99 000	0.18
Nile River	Assouan, Egypt	1 262 000	115 000	0.11
Columbia River	Mouth	259 000	210 000	0.81
Danube River	Mouth	300 000	235 000	0.78
St. Lawrence River	Ogdensburg, N. Y.	383 000	249 000	0.65

* Cons. Engr., Portland, Ore.

The power possibilities, in conjunction with other uses, which may be made of the water of the Columbia River have been studied by a Board of Engineers appointed by the Federal Power Commission. These studies were confined to that part of the river above the mouth of the Snake River which enters the Columbia near Pasco, Wash., and, of this part, there was considered only the main river from Pasco to the mouth of the Clark Fork, a few miles north of the Canadian line; the Clark Fork from its mouth, 20 miles through Canada, then into the United States, to the mouth of the Flathead River, in Montana; and the Flathead River from its mouth to Flathead Lake, also in Montana, the total length along the river channel from Pasco to Flathead Lake being 740 miles. The fall from Flathead Lake to Pasco is 2 570 ft.

Under instructions from the Federal Power Commission, and in accordance with its general policy, the Investigating Board, of which Col. J. B. Cavanaugh, Corps of Engineers, U. S. A., was Chairman, and the speaker, one of the members, made recommendations as to the best uses which might be made of the waters of the Columbia River, and, besides power, irrigation, flood control, and navigation were considered.

In the study, use was made of previous investigations, especially with reference to the probability of practicable dam sites. Long stretches of river on which feasible sites were not known to exist, were assumed as unutilized for power, the results, therefore, indicating minimum rather than maximum or even probable power which can be made available. The figures presented refer only to all-year or primary power, on the basis of nominal flow, which is the flow exceeded or maintained during 90% of the time.

The power possibilities are greatly affected by the large storage available in Flathead Lake (125 000 acres), Pend Oreille Lake (75 000 acres) and Priest Lake (24,000 acres). Practicable storage in these lakes is limited by considerations of local damage to lands and improvements. It was estimated that storage could probably be obtained, as follows:

Flathead Reservoir	1 600 000 acre-ft.
Pend Oreille Reservoir	1 750 000 "
Priest Lake Reservoir	600 000 "
Total	3 950 000 "

Including damages, the storage given can be secured at low cost, probably between $1.00 and $1.50 per acre-ft.

The power possibilities were determined on the principle that all irrigation should have preference as to the use of the water, and allowance was made for the probable future diminutions of flow for this purpose.

As to irrigation, the one outstanding possibility is the reclamation of the Columbia Basin, in the State of Washington, which contains between 1 500 000 and 2 000 000 acres of irrigable land. The project has been investigated by the State of Washington, two diversions having been considered. One is a gravity diversion just across the State line in Idaho, the other is a pumped diversion at the head of Grand Coulee, the pumping to be done by power

developed at an excellent dam site, which has been tested by deep preliminary borings.

These two diversions are mentioned, because a choice between them materially affects the power possibilities. The Board found that the power possibilities aggregated, as follows:

Streams as at present, no storage control..	1 725 000	primary	e.h.p.
All future irrigation allowed for and full storage utilized:			
Columbia Basin irrigated by pumping...	2 443 000	"	"
" " " " gravity.....	1 796 000	"	"

The primary power given as available with a Columbia Basin pumping project is additional to the large amount of seasonal power required for pumping.

The large power possibilities of Columbia River, as given, deal only, as previously stated, with the main river as far as Pasco. Below Pasco, it is possible that 150 000 and 500 000 primary e.h.p. may be made available, respectively, at Umatilla Rapids and Celilo Falls, which may increase the total main river power to about 3 000 000 h.p. A considerable part of the power in tributary streams like the Spokane, Yakima, and Wenatchee Rivers has already been developed, and large additional possibilities remain in these and other streams, notably the Deschutes, Chelan, and Kettle Rivers, all of which is independent of the great potentialities on the west side of the Cascade Mountains.

F. H. TIBBETTS,* M. AM. SOC. C. E.—Mr. Matthes has shown so clearly the scientific application of aerial photography to accurate mapping, that the many uses of simpler airscapes for illustrative purposes are in danger of being overlooked. Aerial photography can be of much assistance in many widely divergent lines of engineering work, both in the field and in construction. Photographs for this purpose are of two general types: Vertical views for aerial mapping, and oblique views ("bird's-eye views") for descriptive aids.

The vertical photographs are taken usually from an altitude of 10 000 to 12 500 ft., with the camera pointing vertically downward. Generally, they have been used in the construction of mosaic maps for military purposes. Maps of the same type have become quite common, especially for city mapping as an aid to the study of transportation, traffic, fire hazard, and similar problems, and for land development, to illustrate cultural conditions such as areas in swamps, improved lands, brush, trees, etc., and to show the position and extent of existing improvements, roads, drainage, buildings, etc.

Work of this kind requires, however, special and precise photographic apparatus not commonly available in the United States, except through cooperation with the military authorities, and is usually too expensive for any except important projects.

Oblique or Bird's-Eye Views.—The oblique, or bird's-eye views, can be taken by any amateur with ordinary commercial apparatus. Photographs of

* Civ. and Cons. Engr., San Francisco, Calif.

this kind are of great value in illustrating preliminary reports, enabling other engineers who have not seen the territory to be covered, and particularly non-technical men interested in such work, to visualize rapidly and accurately the proposed projects. Aerial photographs may also be of great value for illustrating progress reports. Executives, boards of directors, or public officials, are thus enabled to appraise correctly the progress being made in a fraction of the time required to read a progress report, and with much greater accuracy.

In a study of city traffic problems, etc., any practicing engineer may obtain photographs by means of which he can, at his leisure, count automobiles or other units of vehicular traffic, freight cars, boats or buildings, or other improvements. For proposed land development work, aerial photographs show topography in relief, a result not satisfactorily obtained by vertical views, and thus enable those who have not seen the territory to understand correctly such topography, especially in broken or mountainous country. Simple photographs of this kind also show clearly the extent of forest or other vegetation, watercourses, swamps, or lakes, as well as improvements, such as roads, etc.

Good aerial photographs can be taken with any ordinary hand camera that is fitted with a shutter capable of operating at a speed of $\frac{1}{200}$ sec. Under average conditions of good light, probably $\frac{1}{300}$ sec. is better, with which timing, an aperture of F-5.6 to F-8 will ordinarily give properly exposed negatives. Care should always be taken to hold the camera free from direct contact with any part of the aeroplane, otherwise vibrations of the machine are more directly transmitted to the camera, with a corresponding loss in sharpness of the negatives. A camera with a focal plane shutter will probably give better results and a sharper negative than one with the more common, between the lens, type of shutter. Ray filters will seldom improve results, even on a hazy day. The lightest ray filter in ordinary commercial use requires about three times the normal exposure, and under conditions of light that would indicate its use for ground work, an ordinary lens, without the filter, would be nearly full aperture with an exposure of $\frac{1}{50}$ sec. The use of a filter to time the negatives properly under these conditions, would require an exposure so long as to preclude clear negatives. Because they are less sensitive to the vibrations of the machine, plates probably give better results than films, although they are more difficult to use.

With aeroplanes of the more common types, the engineer or photographer occupies one cockpit, with his view somewhat obstructed by the wings or tail of the machine. To obtain good photographs an aviator of experience in this kind of work is required, or one who has been instructed in advance to maneuver his machine so that the object to be photographed can be taken with the camera pointed diagonally away from the machine, about half way between the wings and the tail. The photographs should be taken when the machine is moving straight ahead, as clearer views will thus be obtained. The photographer, by pointing, can indicate the object that he wishes to photograph, and the aviator should then bring his machine into proper position. Ordinarily, the best results can be obtained from an altitude of about 1 000 ft. The limits

for such views, if much detail is desired, are probably between 500 and 1 500 ft.

The direct benefits to the practicing engineer of using an aeroplane for reconnaissance work should never be overlooked. Most engineers have had considerable experience with the use of the camera, and when work of this kind is to be done under their direction, can just as well take the photographs, being the best judges, while in the air, of the subjects which they wish to photograph, and, at the same time, secure the benefit of direct inspection and study of the territory in which they are interested. When an idea of a rough mountainous area is desired, for example, more information can be obtained in a few hours in an aeroplane than in as many weeks of laborious and costly travel on the ground.

The speaker suggests as a matter of general policy that an engineer's duty is ordinarily only half completed when he has obtained a correct solution of the technical problem presented. The remainder of his duty to himself, and particularly to the public, is to convince non-technical, but vitally interested, persons of the merits of the work which he recommends. It is also his duty to keep the public interested and informed concerning the progress of construction work in which it is vitally interested. Aerial photography furnishes one of the most promising fields for informing, educating, and interesting the public in great enterprises. The progress of aviation in America has been most bitterly disappointing to the many men of vision and of courage who have devoted their energy to its advancement. The conquest of the air has been a peculiarly American achievement. Future air supremacy bids fair to be a matter of overwhelming military importance, and the maintenance of a trained aviation personnel is one of the most certain methods of insuring national safety. Pride in a great American invention, far-seeing patriotism, and scientific interest, as well as practical results in enabling the public to visualize and thus appreciate great public improvements, should all combine in enlisting the support of the practicing engineer in the more extended and more general use of the aeroplane and of aerial photography.

Louis C. Hill,* M. Am. Soc. C. E.—Aerial surveys are of benefit when they are supplementary to or supplemented by comprehensive ground surveys. Under many conditions, however, the aerial survey can be of estimable worth when made independent of any ground survey beyond the necessary horizontal control.

In the study of the growth of cities and of the factors affecting congestion, the great detail shown on aerial photographs can be only little more than that indicated on maps of other types. The aerial survey of the business district of Los Angeles, Calif., is a fine example of the results that can be secured. The speaker is informed that remarkable accuracy was obtained.

The real value of the aerial survey is apparent in the study of areas covered by heavy brush, or that are otherwise inaccessible. Under the speaker's direction, such a map was made of the Kern River just north of Bakersfield, Calif., where topographic conditions are such that some form of protection from over-

* Cons. Engr. (Quinton, Code & Hill), Los Angeles, Calif.

flow is necessary to prevent periodic inundation of that city. The overflow areas were covered with dense brush. The channel of the river was divided by many low islands and, in many places, these channels were restricted so as to be almost unrecognizable from the ground. Former channels of which no apparent trace remained, were clearly discernible on the aerial map. The position and magnitude of the canals, the banks of which form the present system of protection, were shown in relation to the various river channels. The effect of obstructions in the river, such as bridges and diversion weirs, became immediately apparent, as well as the amount and character of development on the areas to be protected. All this was shown as it actually existed, and none of it was affected by that indefinite factor, the personal equation. As a consequence, only a relatively small amount of ground surveying had to be done. Horizontal control was made sufficient to fix the scale of the aerial map, and levels were taken along lines projected on that map. Little more detail was needed to supplement the aerial survey.

Another survey is proposed of a large reservoir in an inaccessible region where the attitude of the Indians is such as to make surveying an extremely hazardous undertaking. Certain topographic maps were made several years ago of the probable dam site and of the lower elevations of the reservoir. It is felt that these maps can be extended quite accurately from an aerial survey. This is expected to be especially valuable in the study of the topographic and geologic conditions affecting the location of the dam and the spillways.

C. H. Birdseye,* Assoc. M. Am. Soc. C. E.—Mr. Matthes has very properly brought out the point that a rigid application of the method of utilizing aerial photography is not suitable to the survey of a large area, such as a "quadrangle" as mapped by the U. S. Geological Survey. The topographic maps of the Geological Survey are made in quadrangular units, each of which is an integral part of the map of the United States. These parts must fit together perfectly, and cumulative errors, such as those that are bound to result in surveys not based on accurate geodetic control, cannot be allowed. Since the World War, the U. S. Geological Survey has been using airplane photography as an aid to ground surveys to the fullest extent that its funds and the assistance placed at its disposal have permitted. Its principal difficulty has been in getting the Air Service of the Government to photograph the areas under survey. Limited personnel and equipment have prevented these services from giving the Geological Survey their fullest co-operation. The methods used by the Geological Survey have differed somewhat from those described by Mr. Matthes, but they have been no less successful and have resulted in a large saving in the expense of ground surveys.

Aeroplane photography was utilized in surveying the Schoolcraft Quadrangle, in Michigan, where 273 photographs, covering the area of 220 sq. miles, were taken by the Air Service of the Army. The total time consumed was 5 days, including the round trip from McCook Field, Dayton, Ohio. The actual photographic time, however, was only 7 hours, and the total cost of the

* Chf. Topographic Engr., U. S. Geological Survey, Washington, D. C.

photographic work, exclusive of the salaries of the officers, was $712 or $3.50 per sq. mile.

The ground control had already been established, and the base map projection was made, on which the control points were plotted and the land-line network adjusted before the photographs were used. Herein lies the difference between the method used by the Geological Survey and that of Mr. Matthes.

The photographs were joined together into small mosaics, about four pictures to a mosaic. All features on the photographs that would aid the topographer, such as roads, houses, streams, swamps, fence lines, and timbered areas, were inked on these small mosaics. The mosaics were then bleached, leaving only the inked lines on the scale of the original photographs, that is, 1 : 15 000. These bleached photographs were then reduced by photography to the scale of the base map, that is, 1 : 48 000. These reduced photographs were then adjusted to the control and the land-line net laid down on the base map, fence lines being registered over the section lines plotted on the map. Except at a few points, a perfect adjustment was obtained. The resulting combination of small photographs was re-photographed and printed in non-photographic blue on field plane-table sheets and sent to the field for contour sketching by the usual ground methods.

The cost of preparing these field sheets was about $1.50 per sq. mile, making the total cost of the photographic work less than $5 per sq. mile. The wealth of detail shown in the photographs, such as drainage, fence lines, and timber lines, enabled the topographer to make a great saving in the cost of the ground survey, which was $20 per sq. mile. The cost of office drafting was $3 per sq. mile, making the total cost of the survey $28 per sq. mile. Similar work in adjacent areas by the usual ground-survey methods cost about $40 per sq. mile, which indicates a saving of $12 per sq. mile, or 30 per cent.

The U. S. Geological Survey has also utilized aeroplane photographs in making base maps for the U. S. Bureau of Soils, in extensive surveys of the coast in Santo Domingo and Haiti, and in the revision of old topographic maps, using somewhat the intersection methods described by Mr. Matthes, except that the photographic control obtained from the photographs is tied to carefully executed ground control. The work in Santo Domingo and Haiti was done in co-operation with the Navy Department and the Military Government, and a reconnaissance map of the entire coast of Haiti has been prepared from aeroplane surveys, with no other ground work except the primary triangulation.

Aeroplane photography in topographic mapping will prove of greatest value in the revision of old surveys. It is obvious that a more accurate and complete revision of the culture can be made by the use of aeroplane photographs than by additional field work, which is likely to be only fragmentary.

Efforts are being made to develop a process by which relief can be shown accurately by means of the data obtained from aeroplane photographs. European scientists and the experts of the U. S. Army Air Service at the Experiment Station at McCook Field are working on this problem, but so far with very little practical result. Such application of photographic data may

yet be made, but many years of research and experimental work will be necessary before a method can be developed that will be commercially feasible.

In recent years, many articles on aeroplane photography have been published, and the layman has been led to believe that all that is necessary to obtain an authentic map of a region is to take a few aeroplane photographs and paste them together. Such an operation will give a distorted photograph of the ground, which is properly called a mosaic. It is, however, not a map in any sense of the word, and unless the photographs are rectified and tied to well-established ground control, serious errors in both azimuth and distance will result. Nevertheless, aeroplane photography is the most valuable aid to topographic mapping that has been devised since the plane-table was perfected, and the U. S. Geological Survey proposes to use it to the fullest extent and to assist as much as possible in the development of new methods and new instruments.

EDWARD H. SARGENT,* ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Hogan is to be congratulated for his able résumé of the water power situation in New York State. It may be pertinent to note the availability of the extensive data obtained as a result of the investigations by various State departments during the past fifteen years.

In 1907, John R. Freeman, M. Am. Soc. C. E. (now Past-President, Am. Soc. C. E.), began the first comprehensive study of the water powers of the State, at the direction of the State Water Supply Commission. However, the late George W. Rafter, M. Am. Soc. C. E., as early as 1895, made elaborate studies of the needs and effects of storage reservoirs, tentatively locating many reservoir sites. The investigations begun by the State Water Supply Commission in 1907 were continued until 1911, when that Commission was succeeded by the Conservation Commission. The Conservation Commission continued the work until 1921, when the work was transferred to the Department of the State Engineer, which has since continued the investigations, using therefor appropriations made to the New York Water Power Commission.

The results of the surveys, gaugings, borings, etc., made in the course of these investigations, have been embodied in many reports and pamphlets, most of which are available. Moreover, all the maps, designs, and estimates which have been made, are on file in the office of the State Engineer, and may be examined by any one interested. As a result of the investigations by these various State departments, it is believed that the development of many water power and storage projects will be facilitated.

It is stated on page 737, that the Indian Lake and Salmon River Reservoirs "are the only storage reservoirs of any size, that have been constructed in New York State for power purposes." Mr. Hogan neglected to include the Cranberry Lake Reservoir with a capacity of 3 000 000 000 cu. ft., which is on the East Branch of the Oswegatchie River. This was the first large storage reservoir built for power purposes in New York State, it having been constructed in 1867 by a commission of power owners. Mention also should be made, perhaps, of the Delta and Hinckley Reservoirs which, although con-

* Chf., Bureau of Water Power, Dept. of State Engr., Albany, N. Y.

structed primarily for water supply to the Barge Canal, nevertheless are operated so as to exercise a great beneficial effect on the water powers of the Mohawk River and West Canada Creek. These reservoirs have capacities of 2 750 000 000 and 3 400 000 000 cu. ft., respectively.

WILLIAM T. LYLE,* ASSOC. M. AM. SOC. C. E. (by letter).—To the writer the leading thought in these presentations of different aspects of the Water Power Problem is expressed in the last word of Mr. Hogan's paper, namely, "co-operation". With it might be coupled two others, which he also uses, in order to bring out the nature of the problem and its bearing on the nation-wide problem of general internal improvements. These are "co-ordination" and "conservation", and to them the writer would add still another, namely, "control".

If Engineering is to be defined, as was Civil Engineering in the charter of the Institution of Civil Engineers, to wit, the "art of directing the great sources of power in Nature for the use and convenience of man as the means of production and traffic", this definition must be understood to possess a much broader significance than that attached to it heretofore. The modern engineer utilizes not only the powers and materials of Nature; he utilizes human, social, and legislative powers as well. He is no longer content to concern himself with technical matters alone; he must be more than an administrator of other men's determinations; he must be more than a counsellor and guide; he must possess the well-rounded aggressiveness of the successful business man as well. These qualifications, in the writer's opinion, are especially true of the water power engineer; the water power problem is of such a nature as to develop engineers of this high type. In the interests of National internal betterments, however, the future will probably bring to light engineers of still broader vision and still greater opportunity in the management of the mutually dependent material affairs of the nation.

Co-Operation.—Mr. Hogan has clearly shown the importance of a connected system of water powers whereby a needful and profitable interchange of power can be made between plants of different characteristics; he has pointed out the value of selective development, indicated the nature of a community of interest, and shown how available powers can best be utilized to secure a maximum economic return from each stream.

Co-operation in the water-power field, however, is not enough. The problem should not be limited to the supply of existing demands; the demands also may be created. Where power is available, it should be put to use. This result can be brought about through legislation and appropriation which bring together the producer and the consumer to the great advantage of both. No better way of accomplishing this can be found than in the construction of State and National highways through unimproved country. This applies especially to the Western States where also irrigation, a concomitant of water power development so often is necessary. The extension of the economic limits of high-voltage transmission make possible the opening of new districts regardless of their location with reference to the source of power.

* Prof. of Civ. Eng., Washington and Lee Univ., Lexington, Va.

Co-Ordination.—Power development, irrigation, flood control, water supply, canalization, and National parks are not separate considerations; they should be parts of one big problem and should be dealt with in their mutual dependence as interlocking interests. Independent operations in these several fields are not productive to the general public of a full return for the money expended.

Conservation.—The development of a water power not only secures the power while it can be had, but also operates to conserve the National supplies of coal and oil. As far as conservation is concerned, water power, water supply, and forest preserves may be grouped together. By conserving one resource, the others may be conserved also.

Control.—The harnessing of a stream for power production involves a beneficial regulation in the interests of flood control, irrigation, and of water supply farther down the valley. Among other advantages may be mentioned the stabilizing of channels, improved navigation, better sewer outlets, more advantageous conditions for bridges, elimination of ice gorges, and better heads during flood conditions for low-head power plants on the lower reaches of the stream.

HOWARD R. FARNSWORTH,* ASSOC. M. AM. SOC. C. E. (by letter).—The Act of Congress approved June 10, 1920 (41 Stat., 1063), known as "The Federal Water Power Act" has had more than the usual allotment of troubles in its operation, beginning with the tardy date of its approval and continuing to include the realization that no adequate personnel had been provided in order to make the entire Act practically effective. In this latter respect, the situation appears not unlike that of the "Mineral Land Leasing Act" approved February 25, 1920 (41 Stat., 437), a former Commissioner† of the General Land Office having predicted that no doubt the personnel problem would prove to be one of the weaknesses of the oil-leasing system. The Water Power Act, however, is very comprehensive in its terms and was conceived generally in sound judgment. It is to be regretted that our lawmakers did not give as careful attention to the legal and administrative features as did the engineers to the technical sections, which are excellent. There is much assurance to be found, however, in the fact that the Nation has as the active administrative heads of Federal water-power development and of reclamation by irrigation two engineers of outstanding ability, who are operating in their respective fields with a broad view looking to the early development of the water resources of the country.

In a description‡ of the newly invented 100-kw. triode vacuum tube, Mr. W. G. Housekeeper's important new method of making metal-to-glass seals is explained. Dr. Irving Langmuir who, by means of the construction of a series of five 20-kw. vacuum tubes, used in the place of alternators, recently transmitted radio messages across the ocean, has also discovered new

* Member, Manual Board, General Land Office, Washington, D. C.

† "Oil Land Leasing Law of the United States", by Clay Tallman, *California Oil World*, Vol. XIII, No. 662 (2d Edition, May 26, 1921), p. 78.

‡ By Dr. William Wilson, of the Western Electric Company, in the *Wireless Age* (New York); *Literary Digest*, Vol. 75, No. 8 (November 25, 1922), p. 26.

processes, in conjunction with Mr. J. H. Payne, Jr., of the General Electric Company, and Dr. A. W. Hill, resulting in the development of a 1 000 000-watt (1 340 Eng. e. h. p.) vacuum tube which is intended primarily for the economical transmission of power over great distances. This super-tube which weighs only 60 lb., is described as a two-electrode tube, and it acts as a rectifier of current, and not as an oscillator, also depending, however, on the perfected process permitting the coalescence of glass and metal. Dr. E. F. W. Alexander, Chief Engineer of the Radio Corporation, is quoted in press dispatches as stating that these tubes will soon take the place of alternators in power transmission, and predicting that the transmission of electric power from Niagara Falls to New York City by means of these tubes is a possibility of the future. Dr. Langmuir also states that "it would be rash to predict the limitation of the ultimate use of vacuum tubes in the power field."*

It would be a curious comment if, by the time Congress has amended Section 2 of the Water Power Act so as to provide for adequate personnel in order to make it effective, engineering inventive genius has created the necessity also for the amendment of other sections of the Act in order to provide for new conditions, of possible development by science, in the hydraulic plant.

The preliminary factors of safety essential in every proposed water-power development include the following considerations:

1.—Feasibility of the project:

(*a*).—Physical: Proper physical conditions for the location of the power site; and,

(*b*).—Financial: Satisfactory market load centers at available transmission distances.

2.—Authority to proceed and control of the premises.

The subjects proposed in Section 1 of the foregoing considerations have been adequately covered by the others participating in this Symposium. The writer will discuss the questions raised in Section 2, which will immediately lead him to certain requirements of the Federal Water Power Act, and he will endeavor to set forth the principles, ignorance of which often causes those pitfalls sometimes encountered and which with prompt recognition may be avoided, that are involved in the "control of the premises", as they are construed by the Courts of last resort.

Commerce Power

The Constitution.—Recall, for a moment, the period from May to September, 1787, when the Constitutional Convention sitting at Philadelphia, Pa., under the chairmanship of George Washington, was busily engaged in superseding the inadequate Articles of Confederation in favor of the "Virginia Plan" offered by Edmund Randolph, of that State, which plan, after much debate and many modifications, was finally adopted as the Written Constitution of the United States. One of the radical changes effected in this document was that delegating to Congress full power "to regulate commerce with

* *New York Tribune; Literary Digest,* November 25, 1922, p. 27.

foreign nations, and among the several States, and with the Indian tribes", which is to be found in Article 1, Section VIII, Clause 3, thereof. This clause forms the basis for the distinctions which are drawn in the ownership of the bed and the waters of navigable and non-navigable streams. The subject of commerce continues, after a period of 136 years, to be a paramount issue in Congress as is evidenced by the so-called "Ship Subsidy Bill" (H. R. No. 12817, 67th Congress, 3d Session), now pending and in debate.

In Colonial times, the English Unwritten Constitution, common-law practice, and usage had prevailed in America, and judicial decisions were based exclusively on precedents established in English Courts. The State Constitutions of the United States continue to be construed in the light of this common law, inasmuch as the common law is regarded as having been in existence prior to the Constitutions and as remaining afterward, limited only by such restrictions as the Constitutions impose.*

When, in 1789, the new republic was organized it became necessary to modify the existing principles of jurisprudence and to adapt them to the altered theory of government. The confirmation of John Marshall as Chief Justice of the United States, in 1801, marked an epoch in the history of the Nation, inasmuch as he immediately proceeded to establish the structure of American law on a firm and enduring basis, and for 35 years he bequeathed to the future a great number of valuable decisions, in which the principles of American jurisprudence are set forth with invincible logic and unvarying clearness.

Robert Fulton made his trip up the Hudson River from New York City to Albany, N. Y., in the *Clermont,* in 1807, and, for many years thereafter, this first steamer continued to ply the Hudson, thus revolutionizing the old methods of river navigation. The Legislature of the State of New York had granted an exclusive right to Robert Livingston and to Fulton to navigate the waters of that State "by fire or steam", in consideration of their services in making the steamboat practicable, and in Gibbons *v.* Ogden (9 Wheaton, 1), in 1824, Chief Justice Marshall delivered the opinion of the Court in one of the first pronouncements of the restrictions imposed on State authority, defining therein the power exercised by Congress over interstate commerce and navigation. This decision continues to serve as a standard in the consideration of the questions raised.†

The Relation to Water-Power Development.—In the construction of water-power projects, the dams, head-races, diversion structures, other appurtenant works, and the primary reservoirs, will invade the bed, the banks, and the waters of the stream.

Section (4), Sub-Section (*e*) of the Water Power Act provides, in part, that before the issuance of a preliminary permit to an applicant for a license, the State authorities shall be advised of the proposed action, and publication thereof shall be broadcasted, locally, for a period of eight weeks. Section (5) of the Act states that the sole function of a preliminary permit is to secure

* State *v.* Noble, 118 Ind. 350; Mattox *v.* United States, 156, U. S. 237.

† See, also, Pensacola Tel. Co. *v.* W. U. Tel. Co., 96 U. S. 1; Texas & Pac. R. R. Co. *v.* Interstate Commerce Comm., 162 U. S. 197.

priority to the claimant in his application for a license, and to provide for the necessary examinations, surveys, maps, plans, etc., in order to set forth, clearly, all the conditions which obtain. Section (9), Sub-Section (*b*), of the Act requires the applicant for a license to submit satisfactory evidence so as to show proper compliance with the laws of the State, with respect to the "bed and banks and to the appropriation, diversion and use of water for power purposes," etc. When the power enters into interstate or foreign commerce a certain regulation thereof is stated, in Section (20), to fall within the jurisdiction of the Interstate Committee under the procedure outlined in the Act approved February 4, 1887, as amended (24 Stat., 379).*

It is thus indicated that irrespective of whether the stream is navigable or non-navigable, it has always been necessary for the promoter of a water-power development to proceed in accordance with particular Federal and State laws which apply according to the physical conditions that obtain.†

Factors Determining Navigability

Mr. Merrill, has cited the cases of the Saco, the Connecticut, and the Menominee Rivers in the New England and the North Central States which, although determined on investigation to be technically navigable, were held not to be navigable waters within the definition of the Act, in virtue of the full protection afforded the existing interstate commerce by the laws of the particular States. Federal licenses accordingly were not required for power installations on these streams.

It is fully recognized that the statement of navigability, as set forth in Section (3) of the Water Power Act, regulates the administration of the provisions thereof and that, under Section (23), if the Commission shall find that substantial interests of interstate or foreign commerce are not affected by the proposed construction of a dam or other obstruction across a stream, a Federal license to proceed is not required. This fact, however, should not be confused with the classification and ownership of the public or the State lands involved, in any of the public land States,‡ which rest on the similar conception of navigability, that which is conveyed in the following definition:§

"The settled rule in this country that navigability in fact is the test of navigability in law, and that whether a river is navigable in fact is to be determined by inquiring whether it is used, or is susceptible of being used, in its natural and ordinary condition as a highway for commerce, over which trade and travel are or may be conducted in the customary modes of trade and travel on water."

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* See, also, for this subject, State Freight Tax, 15 Wall. 232.

† For State control over domestic commerce, see Welton *v.* Missouri, 91 U. S. 275; Veazie *v.* Moor, 14 How., 568–574; Cooley *v.* Wardens, 12 How., 299. The police power of States over either domestic or interstate commerce is explained in R. R. Co. *v.* Husen, 95 U. S. 465; Chi Lung *v.* Freeman, 92 U. S., 275; State Freight Tax, 15 Wall., 232; Brown *v.* Houston, 114 U. S. 622; Lake Shore Ry. Co. *v.* Ohio, 173 U. S. 285; License Cases, 5 How., 504; Munn *v.* Illinois, 94 U. S. 113; Commonwealth *v.* Alger, 7 Cush. (Mass.) 53; Lawton *v.* Steele, 152 U. S. 133.

‡ For public land States, see *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 573.

§ The Daniel Ball, 10 Wall. 557, 563; The Montello, 20 Wall. 430, 439; United States *v.* Rio Grande Co., 174 U. S. 690, 698; United States *v.* Cress, 243 U. S. 316, 323; Economy Light & Power Co. *v.* United States, 256 U. S. 113, 121; McGilvra *v.* Ross, 215 U. S. 70, 78; McManus *v.* Carmichael, 3 Iowa 1.

This definition is distinguished in all the cases cited and is enlarged upon in detail, especially in the first two. The quotation itself is taken from the decision of the U. S. Supreme Court in Oklahoma *v.* Texas, United States Intervener, No. 20 Original, delivered on May 1, 1922. The decision is very comprehensive and contemplates all phases of "navigability in fact", ownership of the bed and banks and of riparian rights. It is not yet published in the volumes of U. S. Reports, being found only in the advance sheets thereof, the "Lawyers' Editions", and also in pamphlet form. All the numerous excerpts, hereinafter quoted up to the Section headed, "Meander Surveys and High-Water Level", will be understood to have been taken from this decision, or, where a specific reference to it is made again, the form "66 L. Ed. 444" will be used. The river under consideration in this decision is the Red River, and the details of the suit with photographs of the stream are exhibited in a paper by the writer, entitled "A Review of Important Developments in the Science of Cadastral Resurveys as Executed by the United States Government, with Ethical Discussion Thereof."*

Legal Inference.—Legal inference of navigability, which is sometimes alleged to arise from the action of surveying officers in closing official surveys on the meandered lines of a stream, is held to have "little significance".† A similar inference is not attached to those rivers over which Congress, in permitting the construction of bridges, has added the precautionary provision that there should be no interference with navigation.‡ In like manner, those statements taken from early publications and repeated in later ones, which announce the fact of the navigability of a stream, are not sufficient to establish a legal inference:§

"These statements originated at a time when there were no reliable data on the subject, and were subsequently accepted and repeated without much concern for their accuracy. Of course they and their repetition must yield to the actual situation as developed in recent years."

Decisions of State Courts are also sometimes referred to as determining that a river is navigable in fact: "The United States was not a party and is not bound.‖ There is in the opinion no statement of the evidence, so the decision hardly can be regarded as persuasive here."

The characteristics of a stream are frequently represented so as to establish that the transportation has been and must be exceptional, and that it is confined to the irregular and short periods of temporary high water: "A greater capacity for practical and beneficial use in commerce is essential to establish navigability."¶

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* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 546.

† Also, see Barden *v.* N. P. R. R. Co. 154 U. S. 288, 320; Gauthier *v.* Morrison, 232 U. S. 452, 458; Harrison *v.* Fite 148 Fed. 781, 784.

‡ 66 L. Ed. 444.

§ Also, see Missouri *v.* Kentucky, 11 Wall., 395, 410.

‖ Also, see Economy Light & Power Co. *v.* United States, 256 U. S. 113, 123.

¶ Also, see United States *v.* Rio Grande Co., 174 U. S. 690, 698–699; Leovy *v.* United States, 177 U. S. 621; Toledo Liberal Shooting Club *v.* Erie Shooting Club, 90 Fed. 680, 682; Harrison *v.* Fite, 148 Fed. 781, 784; N. American Dredging Co. *v.* Mintzer, 245 Fed. 297, 300.

Control of the Premises

Navigable Waters.—Although the United States has the power and authority to grant, for appropriate purposes, rights and titles below the high-water mark along and under navigable waters in the Territories* it is its policy not to do so, but rather to leave the bed and waters of navigable streams free from any easement of the upland proprietor.† In virtue of the constitutional rule of equality among the States, each new State becomes, on the date of its admission into the Union, the owner of the navigable waters within its boundaries and of the land underlying the same,‡ an ownership which was reserved and retained by each one of the original States. The character of the State's ownership in the land and in the waters of navigable streams is the full proprietary right, subject, however, to the primary control thereof by the United States over interstate commerce and navigation. Whether a conveyance by any State, of land abutting on navigable streams, confers on the grantee any right or interest in those waters, or in the land under the same, is a matter wholly of local law.§ On such questions, the provisions of the Constitution and statutes of the particular State involved and the decisions of its highest Court are regarded as conclusive.‖

Thus, a State has title to the soil below ordinary high-water mark along navigable streams,¶ and the shore is considered rather a part of the water than as land. The line marking the true mean high-water elevation of a stream is determined from the river bed, and that only is river bed which the river occupies long enough to wrest it from vegetation (Houghton *v.* Railway Company (47 Iowa 370)).** A bank is defined as the continuous margin where vegetation ceases, and the shore is designated as the sandy space between it and low-water mark (McCullough *v.* Wainwright (14 Penn. St., 59)).** The laws of each State, therefore, must be investigated to determine the ownership of the soil below mean high-water level along navigable streams.

There is no uniformity of State laws on this subject. Either of three different rules will be found to obtain, namely, that the riparian owner's title extends to, (1) mean high-water mark; (2) low-water mark; and (3) the middle thread of the current. For example, Section 4529 R. C. of the State of Montana, 1907, provides that:

"Except where the grant under which the land is held indicates a different intent, the owner of the land when it borders upon a navigable lake or stream, takes to the edge of the lake or stream at low-water mark * * *."

On February 25, 1895, in Gibson *v.* Kelley (15 Montana, 417, 422), the State Supreme Court held that rule always to have been the order in Montana. In the State of Michigan, the riparian owner's title is declared to

* Brewer-Elliott *v.* United States, 270 Fed. 100; Irvine *v.* Marshall, 20 How. 561; United States *v.* Winans, 198 U. S. 371; Prosser *v.* N. P. R. R., 152 U. S. 59.

† McGilvra *v.* Ross, 215 U. S. 70–79; Joy *v.* St. Louis, 201 U. S. 332; Scranton *v.* Wheeler, 179 U. S. 141, 190; Kansas *v.* Colorado, 206 U. S. 46, 93.

‡ 66 L. Ed. 444; Scott *v.* Lattig, 227 U. S. 229, 242–243; Pollards Lessee *v.* Hagan, 3 How. 213; Weber *v.* Board of Harbor Commissioners, 18 Wall. 57, and cases cited.

§ Shively *v.* Bowlby, 152 U. S. 1, 48–58; United States *v.* Mission Rock Co., 189 U. S. 391; U. S. Supreme Court in Port of Seattle *v.* O. & W. R. R. Co., January 31, 1921.

‖ St. Anthony Falls Water Power Co. *v.* St. Paul Commissioners, 168 U. S. 349.

¶ Goodtitle *v.* Kibbe, 9 How. 471.

** For detailed definition of bed and banks, see U. S. Supreme Court in Okla. *v.* Texas, etc., January 15, 1923.

extend to the middle thread of the current of the stream, irrespective of whether it is navigable or non-navigable.*

In pursuance of the Montana rule, it would appear that in this and in other public land States where similar laws obtain as to the ownership of the shores along navigable streams, and where the United States becomes a riparian owner to low-water mark, the right of the Government to survey and to dispose of lands up to low-water mark would suggest itself, acting as a trustee for its future grantee. The United States, however, has wisely abstained from extending (if it could extend) its surveys and grants beyond the limits of ordinary high water (Barney *v.* Keokuk (94 U. S. 338) and Frank Burns (10 L. D., 365)).

It is also believed that there is no instance in which the United States after having disposed of the land to ordinary high-water mark has departed from the long-established rule of common law and, afterward, asserted a right to dispose of the space between that and low-water mark (The Mayor of Mobile *v.* Eslava (9 Porter, 578; modified later, in (16 Pet. 234)).

Non-Navigable Waters.—In the case where the United States owns the bed of a non-navigable stream, in disposing of the upland along one or both of its banks, it is, of course, free when disposing of the upland to retain all or any part of the river bed; and, in any particular instance, the question of whether it has done so is essentially a matter of what it intended in the disposal.† Where a contrary intention is not shown by treaty, statute, or by the terms of its patent, the United States "will be taken to have assented that its conveyance should be construed and given effect, in this particular, according to the law of the State in which the land lies."‡

On page 746, Mr. Hogan, has described the difficulties, formerly encountered in the State of New York, in the development of harmonious schemes of power development and operation, on account of the necessity of first obtaining unanimous agreements of private riparian owners relative to the interrupted flow of the streams. He then explains the organization of the River Regulating District which is based on a mutual understanding among all the power owners along a stream.

The common law recognizes certain rights of riparian proprietors in the natural flow of a stream, and it extends riparian ownership along the banks of non-navigable waters to the middle of the stream.§ In this regard, the common-law rule, although modified somewhat, will be found to prevail in most of the States, and ordinarily a riparian proprietor along non-navigable rivers will take to the *filum aquæ*, or the center thread of the stream.

The terms, "middle of the main channel" and "mid-channel," are often used in defining the limit of ownership. When applied to navigable streams

* Butler *v.* G. R. & Ind. R. R. Co., 85 Mich., 246. Also, see, comment on this holding in 159 U. S. 87, 91, 96, relative to the lack of a prior adjudication by the U. S. Land Department in respect of the islands involved.

† Wilcox *v.* Jackson, 13 Pet., 498, 516–517; Irvine *v.* Marshall, 20 How. 558; Gibson *v.* Chouteau, 13 Wall. 92, 99; Utah Power & Light Co. *v.* United States, 243 U. S. 389, 404; Kean *v.* Calumet Canal Co., 190 U. S. 452, 460.

‡ See, also, Hardin *v.* Jordan, 140 U. S. 371, 384; Mitchell *v.* Smale, 140 U. S. 406, 413–414; G. R. & Ind. R. R. Co. *v.* Butler, 159 U. S. 87, 92; Hardin *v.* Shedd, 190 U. S. 508, 519; Whitaker *v.* McBride, 197 U. S. 510, 512, 515–516; and, see, Railroad Co. *v.* Schurmeir, 7 Wall, 272, 287, *et seq.*

§ Middleton *v.* Pritchard, 3 Scammon, 510.

such terms usually refer to the thread of the navigable current, and if there are several, to the thread of the one best suited and ordinarily used for navigation.*

In non-navigable streams, however, which are often without a channel of any permanence or a continuous or dependable flow, "the channel extending from one cut-bank to the other, which carries the water in times of substantial flow * * * was the only real channel and therefore is the main channel."†

Riparian Rights.—The proprietor of lands bordering along the banks of streams also acquires a riparian title to land that attaches thereto by imperceptible degrees as accretions, irrespective of whether the stream is navigable or non-navigable, or whether the additional frontage is the result of the deposit of suspended soil or of the gradual recession of the stream. In preparing the plat of an original official survey from the field notes, the courses of the meander survey are represented thereon as the border line of the stream. It is thus shown to a demonstration, in so far as the original meander survey restricts the title of the grantees of the United States along navigable streams, that the watercourse is the boundary, and not the line of the meander survey at the border of the upland.‡ Where, by action of the water, a river bed has gradually changed, the ordinary high-water mark also changes and the ownership of adjoining land follows with it.§

Portions of the bank of a stream will often be swept away in times of flood after the original survey of the meander lines and after the identification of the surveyed units of disposal abutting them. Some of these surveyed tracts will become thereafter a part of the river bed and others formerly non-riparian will become riparian. In the case of non-navigable streams, where the law of the State confers a riparian ownership to the center thread of the stream, even if tracts which formerly were riparian upland had thus become river bed before their disposal, the proprietorship under the disposal thereof would reach to the middle of the stream, providing a prior disposal had not already been made of the adjacent upland then actually riparian. When, however, the disposal of the tracts thus found to be situated in the river bed is made subsequent to that of the sub-divisions adjoining on the upland back of them, theretofore non-riparian, but which had become riparian, then the conveyance of these latter tracts, as a rule, will include the ownership to the center line of the stream.‖

Meander Surveys and High-Water Level

"Meander lines will not be established at the segregation line between dry and swamp or overflowed land, but at the ordinary high-water mark of the

* Iowa *v.* Illinois, 147 U. S. 1; Okla. *v.* Texas, 66 L. Ed. 444.

† Oklahoma *v.* Texas, etc., 66 L. Ed. 444.

‡ R. R. Co. *v.* Schurmeir, 7 Wall, 272, 286–287; Minto *v.* Delaney, 7 Oregon, 337, 342.

§ Minto *v.* Delaney, 7 Oregon 337, 343; 11 Ohio 314; Steele *v.* Sanchez, 33 NW. Rep. 367; Lockwood *v.* R. R. Co., 37 Conn. 387; Grant *v.* Fletcher, 283 Fed. 245. A gradual change is sometimes equivalent in law to an avulsion, see Kinkaid *v.* Turgeon, *et al*, (109 N. W. Rep., 744) Syllabus.

‖ Oklahoma *v.* Texas, etc., 66 L. Ed. 444.

actual margin of the river or lake on which such swamp or overflowed lands border."*

Ordinarily, all swamp and overflowed lands have been patented to the State under the swamp-land laws (Sec. 2480, U. S. R. S., Act of September 28, 1850 (9 Stat., 519)). These lands are clearly defined in Heath *v.* Wallace (138 U. S. 573, 584). Several varieties of native forest trees are found only within the zone of swamp or overflowed lands. However, all timber growth normally ceased at the margin of permanent water.

"Mean high-water elevation will be found at the margin of the area occupied by the water for the greater portion of each average year; at this level a definite escarpment in the soil will generally be traceable at the top of which is the true position for the surveyor to run the meander line."†

It is not practicable, however, in public-land surveys to meander a stream in such a manner as to reproduce all the minute windings of the true mean high-water elevation, and, in practice, only the general courses and distances of these sinuosities are followed.

Correction of Erroneous Meander Lines.—When the United States has disposed of the lands abutting the meander lines of streams, without reservation in the conveyance, it, as a rule, retains no further jurisdiction over the stream area, irrespective of whether it is navigable or non-navigable in character, provided, however, that the original survey of the meander line was reasonably correctly established.

The Land Department of the Government is charged with the duty of primarily determining what are public lands subject to survey and disposal under the public land laws.‡ It also has the authority to correct erroneous or fraudulent surveys wherever it is proved to its satisfaction that public lands which should have been surveyed, have been left unsurveyed, either as a result of incompetence, inadvertence, mistake, or fraud.§

The Secretary of the Interior, however, in the case of The Marshall Dental Manufacturing Company (32 L. D., 553), and in numerous other departmental land decisions, has announced the policy of the Interior Department to be as follows:‖

"The Department has the power to correct surveys upon a proper showing, but, as has frequently been said, the proper rule is to refuse to disturb the public surveys except upon the clearest proof of accident, fraud, or mistake, where a resurvey may affect the rights or claims of any one resting upon the original survey." (Verified by U. S. Supreme Court, in United States *v.* Lane, January 22, 1923).

* "Manual of Instructions for the Survey of the Public Lands of the United States," Gen. Land Office, 1894, p. 57, and repeated in all subsequent editions.

† Advance sheets of a revision of the "Manual of Instructions for the Survey of the Public Lands of the United States," 1919, p. 213.

‡ Section 453 and 2476, U. S. R. S.; Kirwan *v.* Murphy, 189 U. S. 35; Brown *v.* Hitchcock, 173 U. S. 473.

§ Omagin *v.* Powell, 126 U. S. 691; Horne *v.* Smith, 159 U. S. 40; French Glenn Livestock Co. *v.* Springer, 185 U. S. 47; Miles *v.* Cedar Point Club, 175 U. S. 300; Tubbs *v.* Wilhoit, 138 U. S. 261; C. & D. Lumber Co. *v.* St. Francis Levee District, 232 U. S. 186; Gauthier *v.* Morrison, 232 U. S. 452; Producers Oil Co. *v.* Hansen, 238 U. S. 325; Lee Wilson & Co. *v.* United States, 214 Fed. 630; 227 Fed. 827; affirmed by U. S. Supreme Court, November 5, 1917; U. S. Supreme Court in Jeems Bayou F. & H. Club, *et al.*, *v.* United States, January 2, 1923.

‖ This policy was also applied on October 30, 1914, in a ruling on an application for survey by the Klickitat White Pine Co. (Duluth 011106).

Although the long-established rule, to the effect that the actual physical corners and monuments of an original survey hold precedence over the positions thereof as indicated in the record field notes, is quite general in its application, an instance will occasionally arise where an absurdity or a fraud is developed in the strict application of such a principle; in which case, the courses and distances of the original record may hold over the original monuments.* If such a case should arise or other erroneous conditions are developed in the identification of a false original meander survey, the meander lines may then be held to have become a strict boundary. If one should desire to investigate the degree of accuracy of the early meander surveys along any stream, for the determination of a possible irrelation when compared with a true mean high-water elevation which might clearly have existed, as such, at the time of the original survey, the method of traverse adjustment will be utilized for this comparison, such as is explained by the writer in his paper, previously mentioned, covering the practice of the Cadastral Engineering Service of the General Land Office.†

The Survey of Islands.—Artificial lakes and reservoirs, as a rule, are not segregated from the public lands, however, in the original survey of the mainland fronting on any non-navigable body of water, all islands opposite thereto above mean high-water elevation are subject to survey and disposal.

"Even though the United States may have parted with its title to the adjoining mainland, an island in any meandered body of water, navigable or non-navigable, known or proven to have been in existence above the mean high-water elevation at the date of the admission of a State into the Union, and at the date of the survey of the mainland, if omitted from said original survey, remains public land of the United States, and, as such, the island is subject to survey."‡

If, however, an island is formed on the bed of a navigable river subsequent to the date of the admission of the State into the Union, the title to such island is vested in the State and not in the United States.§

Extension surveys, as described in the foregoing, which are executed for the purpose of embracing public lands along bodies of water held to have been erroneously omitted from the original survey, are generally not undertaken by the General Land Office until after the receipt of a formal application therefor, a careful examination of the land in question, and proper authorization by the Secretary of the Interior to proceed.

Consequences of Indiscretion

It is not the purpose of this discussion to alarm the promoter or the owner of any water-power project, inasmuch as all the laws which relate thereto are

* Security Land & Exploration Co. *v.* Burns, 193 U. S. 167–179; Ainsa *v.* United States, 161 U. S. 208, 229; White *et al. v.* Luning, 93 U. S. 514–524; Shipp *et al. v.* Miller's Heirs, 2 Wheat. 316.

† *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), pp. 544, 545, Figs. 9 and 10.

‡ Advance sheets of a revision of the "Manual of Instructions for the Survey of the Public Lands of the United States", 1919, p. 217. See, also, McManus *v.* Carmichael, 3 Iowa 1; Wiggenhorn *v.* Kountz, 8 Amer. State Rep. 150; Hardin *v.* Minn. No. Ry. Co., 84 Fed. 287; in which it was shown that the Government had treated the islands involved as separate property.

§ Pollard *v.* Hagan, 3 How. 212; Widdicombe *v.* Rosemiller, 118 Fed. 295; Hardin *v.* Jordan, 140 U. S. 371.

administered with the broad view of encouraging such developments. It is, however, the part of wisdom to know the premises involved and to establish, definitely, the control thereof before proceeding with the expenditure of large sums of money in project construction, always remembering, where the case may apply, that a naked, unlawful trespass cannot initiate a right to any part of the public domain,* and that "possession not based on a legal right, but secured by violence and maintained by force and arms, cannot furnish the basis of a right enforceable in law."†

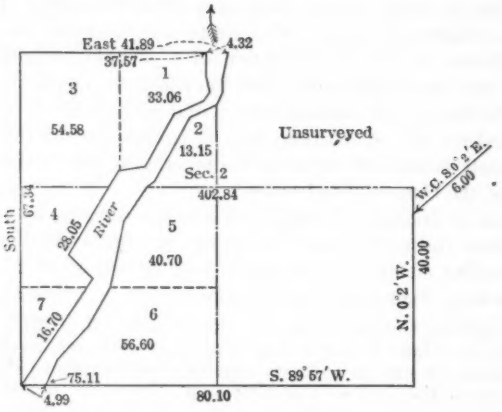


FIG. 30.

With the foregoing principles in mind, attention is invited to Figs. 30 and 31, wherein is portrayed the situation now confronting a certain private company which owns a completely installed power unit of some magnitude, one which is locally well known. The physical conditions exhibited in Fig. 31 are identical with those which are represented as existing. All names, however, and other means of identification are fictitious and are purposely so on account of the pendency of the case.

This power installation was completed in 1907, whereas the control of the premises was not adequately considered until 1922. The company holds a fee-simple patent for Lots 1, 2, 3, 4, and 5, Section 2, in the position as shown on the plat of the original survey, Fig. 30. The dam has been placed in the natural position for it at the outlet of a natural basin and at the entrance to a narrow canyon which is formed by the river. In this position, however, the east abutment may possibly be found to rest on extant public lands of the United States, which are subject to survey and disposal under the public land laws.

If the land lying along the meridional mid-section line of Section 2, inside the erroneous original meander survey, and between the north angle-point of the original survey of Lot 2 (see Fig. 31) and the true mean high-water level

* Swanson *v.* Sears, 224 U. S. 182.
† Lyle *v.* Patterson, Syllabus, 228 U. S. 215.

of the river, is determined to be vacant public land of the United States, then the only apparent possibility that the east abutment of the dam might rest on the company's property, would lie in the circumstance that the exterior side line of the riparian addition to Lot 2 which extends westerly to the center of the stream, might fortuitously intersect the east bank at a point north of the dam.

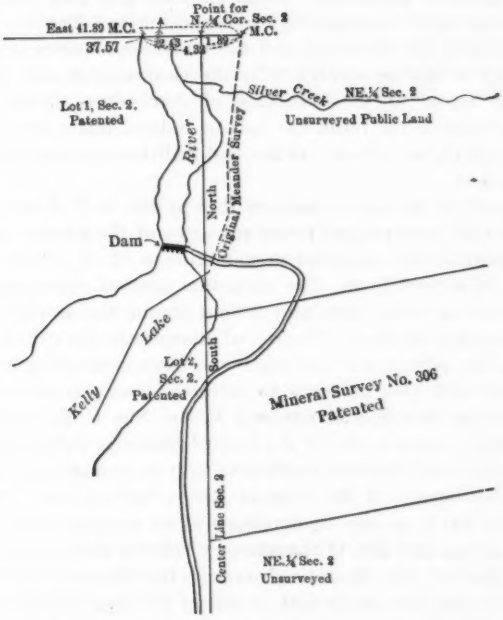


FIG. 31.

If the company has not already the necessary ownership, it may acquire the title through either a special Act of Congress, or by proceeding under the various land laws, or, if an application for a license is filed, under Sections 21, 23, and 24 of the Federal Water Power Act, immediate protection will doubtless be obtained. It is emphasized, however, that the initiative in all these actions rests with the power company. In this particular case, the past record of apparent indifference to security does not indicate a high degree of understanding of such matters, or of initiation. Instances of similar imprudence may exist elsewhere, which are not realized by the parties most interested. These cases may develop into situations of much greater complexity, with reference to the inception of private adverse rights, before they receive even ordinary consideration.

A fully developed and secured water-power plan is the greatest of all measures for the conservation of natural resources, inasmuch as perpetual benefits are made available thereby. The enormous quantities of coal. fuel oil, and all

other forms of fuel now being expended in the generation of steam, may be saved by the hydro-electric power developed, irrigation of agricultural lands is made possible by the use of the water leaving the power wheel, and the possibility of destructive floods is reduced through the storage and control of water in the reservoirs.

The potential water power of all the streams of the United States is not a matter of accurate knowledge. Total amounts will vary widely according to the particular basic assumptions of each estimate. The most comprehensive survey of both the developed and the undeveloped water-power resources of this country is that accomplished by the Government, and which is to be found in the report of the Secretary of Agriculture dated January 20, 1916.* This report is the result not only of independent corrective investigations made in 1915, but also of studies and additions accomplished by several Federal agencies.†

From page 17 of the report made in 1916 by the U. S. Forest Service, one finds that the total undeveloped power resources of the streams of the United States is conservatively estimated at a minimum of 27 943 000 h. p., and a maximum of 53 905 000 h. p. The minimum amount represents continuous power at periods of lowest flow, and the maximum, the average power available during one-half the time. The use of storage was not considered in these estimates, and the efficiency of the water-wheel was assumed at 75 per cent.

In contrast with this amount, an estimate based on other assumptions is that repeated by Mansfield Merriman,‡ M. Am. Soc. C. E., which states that the total potential water power of the United States is 200 000 000 h. p. Mr. Hogan estimates that 2 000 000 continuous h. p. represents one-half the undeveloped potential energy of the Niagara River; various other engineers consider that 4 000 000 h. p. may be developed at an economic cost in the State of California alone, and Mr. O'Shaughnessy believes that a close estimate of all the possibilities of that State lies between 8 000 000 and 9 000 000 h. p.

A statement§ has been made that, in one of the large steam plants of New York City, each horse-power developed now costs about $50 per operating year. If this estimate is only approximately correct, it is fully realized that the development of the power resources of the streams is the big problem, the answer to which represents one of the most impending needs of this country. The enactment of the Federal Water Power Act is a long step toward the solution of this problem.

J. W. Swaren,‖ M. Am. Soc. C. E. (by letter).—These papers cover every phase of the water-power problem, except the obtaining of funds and the relation of the investing public to programs of extension.

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* Senate Document No. 316, 64th Congress, 1st Session.

† Report, in 1908, of the Bureau of the Census to the National Conservation Commission (Senate Doc. No. 676, 60th Cong., 2d Session); report, in 1908, of the U. S. Geological Survey, also submitted to the Conservation Commission (*Water Supply Paper No. 224*); report of the Commissioner of Corporations, dated March 4th, 1912; and the report, in 1916, of the U. S. Forest Service (Senate Doc. No. 316, *supra*).

‡ "Treatise on Hydraulics," 8th Ed. (1907), p. 371.

§ Dr. Edwin E. Slosson, of Science Service, in its *Daily Science News Bulletin* (Washington, D. C.); *Literary Digest*, Vol. 75, No. 9 (December 2, 1922), p. 25.

‖ Engr., Bureau of Internal Revenue, Washington, D. C.

In his paper on the "The Prospective Competitor Method of Valuation of Property",* M. L. Byers, M. Am. Soc. C. E., discussed certain features of the rights of existing investors. Of equal or greater import are the rights of investors in new projects. They should know that the market is large enough to absorb the new unit which their money is developing and that the future growth of revenue-producing load will not be outstripped by increased competitive development, or, perhaps, completely dissipated before the net revenues wipe out the early operating deficits.

Rate-making bodies are not inclined to look with favor on rates sufficiently high to re-absorb such deficits. Yet such rates are as real as interest on bonds during the construction period and, on the basis of present worth, as amenable to capitalization.

On the other hand, programs on scales of insufficient size are equally likely to financial disaster. Recently, the writer has observed a semi-industrial power station of the usual efficiency, which has, as its only outlet, sale of power to an existing network with large station capacity. The maximum block of power which the new station can contract, can only be marketed at a price lower than operating expenses. As a result, more than one-half the station is idle, whereas a station of double the size would have a market, and the distributing organization would not be under the necessity of increasing its generating capacity, as it now is.

Mr. Merrill states that certain activities are not undertaken because of lack of funds and insufficiency of personnel, more particularly appraisal work. The writer would like to draw his attention to a rapid method of appraisal that is proving successful where appraisals are necessary in determining income taxes. The books of the company are taken as a basis—costs being checked by auditors—and a brief physical inspection is made by an engineer to determine the general conditions of the original construction and of present-day replacement methods. Factors based on general economic studies are applied to determine present-day physical values. An economic study is then made of the particular company, and curves of expectancy are developed. Net profits actually received in subsequent years are proving that accuracy is being obtained by these methods of economic valuations.

Amplifications of Kelvin's law have been used with success by the writer for economic predeterminations. Its application to the solution of general engineering economics offers little more complication than its application to the economic determination of a copper network.

The investor is entitled to a careful study of the market which is to be served, as well as how it is to be served, preferably by a disinterested investigator, with a continuing experience of wide scope. The writer hopes that Mr. Merrill had such studies in mind when he spoke of important, but not yet undertaken, duties. He states that the Federal Commission is concerned largely with the enforcement of a contract obligation. Protection of the general public, of which he speaks, requires that the contractor be capable

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1313.

of fulfilling his contract throughout its life. The necessary investigations to determine those features already assumed by the Commission should supply most, if not all, the data to enable a satisfactory determination of expectancy.

Water-power securities should be sound under all conditions of general economic stress. Soundness is only possible when the market is assured for the life of the security at the least. This can be predetermined by engineering methods, and should be checked as carefully as any other part of the design.

J. P. JOLLYMAN,* ESQ.—Some of the phases of high-voltage transmission and some of its problems have been discussed interestingly by Mr. Peek and Professor Ryan. The speaker will discuss this subject from the standpoint of operating companies which are confronted with increasing demands for electric energy, and are finding their sources of such supply being utilized more and more in the regions near the load. They are compelled, therefore, when dependent on hydro-electric energy, to reach farther and farther from their load centers for their supply. With this necessity for transmitting energy over greater distances, they are forced—if they want to keep their costs down—to the use of higher voltages. The economic situation in California has forced the operating companies to the use of 220 000 volts as the most economical solution of their problem.

The question of distribution of large amounts of hydro-electric energy in other parts of the United States has been discussed in connection with this subject. The water-power developments of Niagara, and the possibilities of the Colorado and the Columbia Rivers have been mentioned. However, the utilization of those powers and the distribution thereof to the places where there will be a demand for them will be dependent on the use of the highest transmission voltages.

The operating company, when confronted with the necessity of using a voltage higher than that which has heretofore been utilized, finds itself dependent on knowledge that has been acquired comparatively recently. There is no long period of experience with the highest operating voltages. The operating companies are just arriving at that stage, and they are dependent on the technical knowledge that is acquired by the researches made at the universities, of men like Professor Ryan, and his students. The operating companies can obtain experience with these voltages only after they have them in service. They cannot and must not invest millions of dollars in something that is unknown or untried. They must take a step in advance, they must be dependent on such knowledge as has been accumulated by recognized authorities.

In behalf of the operating companies, the speaker wishes to express his appreciation of the time and money that has been expended in research by the manufacturing companies and the universities. Only by such means is it possible for the people of the country at large to enjoy the benefits of continually increasing supplies of low-cost energy which in California, at least, must be derived from water power and must be transmitted over continually increasing distances.

* Chf., Dept. of Eng., Div. of Hydro-Electric and Transmission Eng., Pacific Gas & Elec. Co., San Francisco, Calif.

F. P. WILLIAMS,* M. AM. SOC. C. E. (by letter).—Considering the test that the electric power business has been passing through during the last few years, in the matter of taking care of both the increased demand and the maintaining of rates when everything has advanced in price, it is of especial interest to note in Mr. Hogan's paper the tendencies of water power development at the present time, and their possible effect on future conditions.

A study of a score of scattered public service companies operating in New York State during the period from 1910 to 1920, shows that the average revenue per kilowatt-hour for all classes of service, has remained substantially the same. It is believed that the companies are representative, and that the showing noted is typical. This maintaining of rates has happened in the face of the rise in cost of practically everything since 1914.

An important factor that has helped to secure the record, especially in regard to rates, is the increase in the proportion of water to steam power. In 1910, the energy generated by companies reporting to the Public Service Commission, outside of New York City, amounted to a total of 950 000 000 kw-hr. In 1910, the hydro-electric energy generated was 4.4 times the amount of steam-generated energy; in 1920, the total generated was 3 200 000 000 kw-hr., or 3.4 times the amount generated in 1910. In 1920, the hydro-electric energy generated was 5.2 times as great as that generated by steam, indicating an increase of 0.8 in the ratio of hydro-electric over steam power. The present tendencies, pointed out by Mr. Hogan, indicate how additional power may be available through economies in practice, and by further development. The result will probably be an increase in the ratio of hydro-electric power to steam power. Although the conditions just noted have obtained in the operation of public service corporations, private steam plants have had to face more pronounced increased operating costs, especially of fuel. It is of still greater interest, therefore, to such power users that additional hydro-electric power be developed.

In view of the power conditions that have obtained during the past few years, it is very desirable that the tendencies outlined by Mr. Hogan be followed for the purpose of maintaining that record and improving it, if possible.

Although the fact that the ratio of water power has kept pace with the steady, annual increase in the amount of power required, has increased over the steam power, and has been of great value in maintaining rates, there have been other economics helping toward the same end, such as improvements in machinery, greater efficiency in transmission, greater general refinements in practice, and changes in control, that have contributed to that end.

That an increased proportion of water power has not been the sole element tending to keep rates per kilowatt-hour from rising, is shown by the lowering of rates during the past few years in New York City, a steam-generating center. The average revenue per kilowatt-hour, as reported by the public service electric companies, except street railway companies of New York City, in 1908, was 6.9 cents. In 1919, the average revenue was 4.4 cents. This is all steam-generated energy. In 1919, the amount sold was 1 104 466 493 kw-hr.

* Secy., New York Water Power Comm., Albany, N. Y.

In the case of the city plants, the coal required to generate 1 kw-hr. in 1908 was reported to the Public Service Commission as 4.20 lb. In 1919, it was 2.38 lb., or six-tenths of the former quantity, showing a much greater economy in the use of coal.

Although a great deal has been accomplished in keeping down costs, by invention and refinements in operating, it does not appear as safe to rely on economies in this direction in the future, as to rely on following the present tendencies outlined by Mr. Hogan, especially in securing greater economies through the construction of storage reservoirs, the inter-connection of service, etc. For example, the present developed hydro-electric power may be trebled by the increase in power that may be secured in present plants through river regulation, and by the construction of plants to use the undeveloped power.

Storage reservoirs for regulating stream flow hold promise of great general benefits. Regulation will result in lessening flood damages, in increasing power output, and in improving navigation in the large streams by increasing the low-water flow. However, when it is suggested that storage may be carried to a point that will make Adirondack powers independent of steam, if desirable, it must not be overlooked that, in the case of this power, which requires to be transmitted a long distance to market, auxiliary power will always be desirable, if for no other reason than to protect against interruption of service, to which all long-distance lines are subjected.

As indicated, the conservation of the natural resources of New York State has been studied carefully, both by public and private authorities. Stream flow records, forming the scientific basis for any development, have been generally maintained. Although the power record of the past few years has been a noteworthy one, there appears to be no doubt that the future will give a good account of itself, if the tendencies outlined are advanced rapidly enough to meet conditions. Fortunate, indeed, is the State that has ample coal deposits. Happily, by the advancement made in hydro-electric art, and by the careful study and development of their water powers, other States may approximate the coal advantage of their sister States.

G. B. Herington,* M. Am. Soc. C. E. (by letter).—Mr. Matthes' paper is a valuable contribution to the literature of applied reconnaissance. The use of the aerial photographic method in river and power surveys as described has a peculiar value not only in making available a base map that is directly visible and interpretable as the eye sees the subject, but it also has a great advantage in speed and thoroughness.

Recent articles, in the technical press, on aerial mapping in cities have brought out this same feature, that is, making clearly and naturally visible an infinite amount of related detail, without conventionalizing. Mr. Matthes describes the method and its application on an actual engineering reconnaissance problem of a high order. Further interest would be added if in his closure he would discuss more fully the use of the stereoscope and the manner in which it functions, as this seems to be a very interesting feature of this method of mapping.

* Portland, Ore.

One of the greatest difficulties in reconnaissance usually is the necessity of continuous discrimination—due either to lack of time, funds, or force—between what should be mapped and what may be mapped in a country which may be under investigation primarily for determining whether any further expenditure for surveys is justified. The slower hand methods of surveying a vast stretch of country nearly always must be applied so that, should further work be desirable, the original survey may serve as a base for its extension, thus avoiding repetition and waste. Where reconnaissance is made by riding or walking and taking such observations as may be warranted, the investigator nearly always has a feeling of limitation on account of the personal horizon. He feels that he should cover a great deal more ground than he can, in order to eliminate, by positive conclusions due to actual examination, any possibility of errors of omission.

Where reconnaissance is reduced to any form of line or stadia measurement, the investigation of large areas may become expensive. It frequently happens that reconnaissance must be made hurriedly, and only as complete a report rendered as will be permitted by the time and funds available and the character of the territory to be covered.

Time and financial limitations which have hampered work of this kind seem to be lessened or eliminated by the aerial method.

During the winter of 1919-20, a tornado worked havoc in certain timbered areas along the coastal slope of the Olympic Peninsula in Western Washington. This tornado seemingly ricocheted over the vast timber area, recurrently dipping into the timber and then rising above it, along about 100 miles of its course. The earliest attempts to get a preliminary estimate of the damage disclosed that the trails were blocked by fallen timber and that in some places the tangle of timber trunks and tops was so great that experienced woodsmen and cruisers had difficulty in making a mile a day across some of the damaged areas. In determining the approximate damage, a large force of cruisers was required for practically the entire following year.

A large part of this territory is United States National Forest, and the executives of the Forest Service decided at an early stage that aeroplanes equipped for photographing could give the earliest general information on the damage. Arrangements having been made for this service, photographs of a series of strips were taken along the course of the storm. Although some fog was encountered, the photographs showed the location and nature of the damage quite thoroughly. It is hoped that the Forest Service will make public more of the data on the time, result, and cost of this work.

The writer has examined the photographs and has found them peculiarly interesting. The topographic features of the country were thoroughly determinable, and the different kinds of trees could be clearly distinguished. Although the flights were made at a higher altitude than that used in normal mapping, the crowns of the trees could be counted in the photographs.

The more progressive logging companies make their plans from topographic maps prepared mainly from barometric observations taken during the close cruising of the timber, the contour intervals being from 10 to 20 ft., and

usually 50 ft. Control levels are generally run along section lines, the contours being sketched as the areas are traversed by the cruisers.

In studying Mr. Matthes' paper, the writer has become impressed by the fact that the aerial method of photographic mapping should offer a wide field in future timber land surveys, particularly those made prior to the opening of new timber land areas for operation. The strip counting of actual trees, which is done in ordinary cruising, may be thus modified to some extent, and the cruising tend more to become an examination of quality and of the average size of the trees, as this cannot be determined from the air. An actual tree count of a territory is thus rendered practicable, and the shape relation and direction of topographic features are determined automatically, and the location of corners and land lines, and their position on the photograph is possible in any one of several obvious ways. The necessary surveys can then be best made on photographically pre-determined directions, instead of being run over section lines, as at present.

The present methods of timber cruising, however, will probably be continued without change, because certain practices have been developed, and the language of the industry and the training and special skill of the cruisers is dependable and understood. The topographic mapping, however, may well be removed from their work, which would then be more that of judging the quality and quantity of timber, in view of the possible adaptation of the better method of base mapping which aeroplane photography seems to offer.

The writer has long contended that base levels for contour work in timbered areas should be run along controlling ridges and streams. This would usually require much less leveling, and the controls would be where they were needed. The contour surveying, thus becoming a separate function, could be done, preferably, in advance of or after the cruising of the timber stand, and thus become a groundwork for the subsequent railway surveys and logging layouts.

The photographic mapping of an area would give a fundamental base, show detail far beyond hand-mapping possibilities, provide the base for the actual surveys necessary in determining elevations, and could be done in much the same manner as Mr. Matthes describes for the contour work on the Tennessee River survey.

The paper gives a cost of about $61 per sq. mile on a territory about 80 lin. miles in extent. Timber cruising generally costs from 30 to 55 cents per acre, under contract, in the Northwest, for thorough standard work. Topographic work, as now done, adds to the cost about 20 to 35 cents per acre, thus making the range of present cost 45 to 85 cents per acre. It should be possible on smaller areas to furnish basic photographic maps at $100 to $150 per sq. mile, or at 15 to 20 cents per acre, cruising at 40 cents per acre, with contour survey control at 20 cents per acre. It will not make a much more expensive mapping process, but will furnish a vastly more comprehensive and valuable basic mapping of the territory under study, as judged by standards of visibility, ease of reading, and its actual portrayal of conditions.

Logging involves the application of sound engineering, planning, and execution, by men of sound common sense and experience who have grown up in the work, or by technically trained engineers. The fundamental preknowledge of the problems confronting the operator, as conveyed by its mapping, therefore, requires the best possible standard of informative data and visualization. The writer believes that the application of aerial photographic mapping of timbered areas, properly done, will be of great benefit, as reflected in operating and air survey costs and general results obtainable.

The field for this method in irrigation and power investigation work is enormous. The fact that an irrigation project covering a vast area can be quickly and informatively portrayed on a photographic map which will show infinite flat detail and locate the topographical elements of the whole area, will act as a base for planning all the controls and main final survey features which must be determined accurately, and will be justified ordinarily if used for no other purpose. Heretofore, no other method has been available for getting so quickly and accurately a topographic problem in large areas before the engineer and the financier, who must visualize the whole problem. One of the greatest values, in such reconnaissance, is the portrayal of the things that are not required to be done, and the better application of effort. Although it has been well known that aeroplane mapping has been done, and much progress made, the publication of tested and workable methods, as applied on an extensive survey, has marked a starting point for the extension of the methods used.

W. G. Hoyt,* Assoc. M. Am. Soc. C. E. (by letter).—The U. S. Geological Survey has recently filed for public inspection at its office at Washington, D. C., and at the offices of its District Engineers at Boise, Idaho, and San Francisco, Calif., manuscripts of four reports as follows: Preliminary Report on Power Possibilities of Snake River between Weiser and Lewiston, Idaho; Water Power Resources of Salmon River between Salmon and Its Mouth; Water Power Resources of Snake River between Milner and Weiser, Idaho; and Klamath River and Its Utilization.

These reports were prepared by hydraulic engineers of the U. S. Geological Survey, the three first named by the writer and the last named by E. C. LaRue, M. Am. Soc. C. E. The reports are based on extensive field investigations and office studies and contain detailed information relative to sites at which 2 000 000 h.p. could be developed at times of low water.

Snake River Between Milner and Weiser†

1.—The geologic conditions at most of these sites will permit dams to be constructed to the height of economic feasibility.

2.—The flow from springs makes almost the entire stretch available as a source of power regardless of any future diversions for irrigation.

3.—Practically all the power available can be developed without interfering materially with existing structures or with any other proposed development.

* Hydr. Engr., U. S. Geological Survey, Washington, D. C.

† See Table 19.

TABLE 19.—UNDEVELOPED POWER IN SNAKE RIVER AND ADJACENT SPRINGS BETWEEN MILNER AND WEISER, IDAHO.

No.	Name.	Scheme.	Miles up stream from Enterprise.	Head, in feet.	Flow, in Second-Feet.				Horse Power.			
					Present.		Regulated.		With present flow.		With regulated flow.	
					90 per cent. of time.	50 per cent. of time.	90 per cent. of time.	50 per cent. of time.	90 per cent. of time.	50 per cent. of time.	90 per cent. of time.	50 per cent. of time.
12GR 1	Dry Creek Falls	A	199.7	136	300	5 500	400	400	3 280	60,000	4 300	4 300
12GR 1	" " "	B	199.7	161	300	5 500	400	400	3 900	71 000	5 100	5 100
12GR 2		..	189-196	150	425	5 600	550	550	5 100	67 000	6 600	6 600
12GR 3	Twin Falls	..	185.6	205	550	5 600	730	730	9 000	92 000	12 000	12 000
12GS 1	Auger Falls	A	174.6	154	860	5 900	900	1 000	10 600	78 000	11 100	12 300
12GS 1	" "	B	174.6	140	860	5 900	900	1 000	9 600	66 000	10 100	11 200
12GS 2		A-B	165.2	28	1 750	6 800	2 000	2 100	3 900	15 000	4 500	4 700
12GS 3	Devils Washboard	A-B	162.4	30	1 950	7 000	2 200	2 300	4 700	16 800	5 300	5 500
12GS 4	Kanoka Rapids	A	160.0	31	2 500	7 700	2 750	3 000	6 200	19 000	6 800	7 450
12GS 4	" "	B	160.0	66	2 500	7 700	2 750	3 000	13 000	40 500	14 500	16 000
12GS 4	" "	C	160.0	86	2 500	7 700	2 750	3 000	17 000	53 000	19 000	20 500
12GS 5	Upper Salmon Falls	..	149.5	80	5 000	10 000	5 000	5 300	32 000	64 000	32 000	34 000
12GS 6	Clear Lakes	A	161.2	170	400	400	400	400	5 400	5 400	5 400	5 400
12GS 6	" "	B	161.2	90	450	450	450	450	3 200	3 200	3 200	3 200
12GS 7	Box Canyon	..	156.3	132	330	330	330	330	3 500	3 500	3 500	3 500
12GS 8	Bickel Springs	..	152-153	150	275	275	275	275	3 300	3 300	3 300	3 300
12HA 1		..	127.6	130	6 400	11 400	6 400	6 800	67 000	119 000	67 000	71 000
12HA 2		A	124.0	30	6 450	11 500	6 450	6 850	15 500	27 600	15 500	16 500
12HA 2		B	124.0	80	6 450	11 500	6 450	6 850	41 500	73 500	41 500	44 000
12HA 3		A	118.3	40	6 500	11 500	6 500	7 000	21 000	37 000	21 000	22 400
12HA 3		B	118.3	70	6 500	11 500	6 500	7 000	36 000	64 000	36 000	39 000
12HA 3		C	118.3	120	6 500	11 500	6 500	7 000	62 000	110 000	62 000	67 000
12HA 4	Crane Falls	..	72.4	30	6 800	12 000	6 800	7 300	16 300	29 000	16 300	17 500
12HA 5	Halls Ferry	A	66.3	40	6 800	12 000	6 800	7 300	22 000	39 000	22 000	23 500
12HA 5	" "	B	66.3	75	6 800	12 000	6 800	7 300	41 000	72 000	41 000	44 000
12HC 1	Crocker*	..	....	...	.....	.....	.....	.....	.....	.....	.....	.....
12HC 2	Guffey	..	13.5	27	6 900	12 500	6 900	7 500	15 000	27 000	15 000	16 000
.....	Lower Salmon Falls	..	....	15†	5 100	10 100	5 100	5 400	6 100	12 100	6 100	6 500
									250 700	724 100	260 700	276 350

* Not considered feasible.
† Increased head possible at Lower Salmon Falls by the construction of a new dam and power house.

4.—Surveys and studies made at 18 sites show that the undeveloped horse-power available at 70% efficiency is as follows:

Without regulation of flow:	
Available 90% of the time.....	250 700 h.p.
Available 50% of the time.....	724 000 h.p.
With regulation of flow (all flow above Milner to be used for irrigation):	
Available 90% of the time.....	260 000 h.p.
Available 50% of the time.....	276 400 h.p.

Snake River Between Weiser and Lewiston, Idaho*

1.—On Snake River between Huntington, Ore., and Lewiston, Idaho, there is enormous undeveloped power, which may be utilized without interfering with the use of the water in the upper stretches for irrigation.

2.—Most of this undeveloped power lies between Homestead, Ore., and a point 12 miles up stream from Grand Ronde River. Between Huntington and Homestead, the railroad and, to some extent, the geologic features, prevent any large development of power. In the stretch above Lewiston, the geologic features and the small slope of the river make the possibility of developing power extremely remote.

3.—Owing to the inaccessibility and the nature of the canyon through which the river flows and to the great floods to which the river is subject, the conditions are not favorable for the development of what at present is considered cheap power.

4.—The total undeveloped horse-power at 70% efficiency is as follows:

Without regulation of flow:	
Available 90% of the time.....	919 000 h.p.
Available 50% of the time.....	1 515 000 h.p.
With regulation of flow (all flow above Milner used for irrigation):	
Available 90% of the time.....	800 000 h.p.
Available 50% of the time.....	1 130 000 h.p.

Salmon River Between Salmon and Mouth†

1.—Its well-sustained flow, its exceptionally great slope in view of its size, and the numerous well-spaced sites along it, make Salmon River a valuable source of power.

2.—The flow of the river may perhaps be increased by about 500 sec-ft. for nine months of each year by constructing one reservoir site.

3.—Twenty-five well-spaced dam sites have been located and surveyed.

4.—The distance from a railroad, ranges from 30 to 102 miles, and the cost of railroad construction would be high.

* See Table 20.
† See Table 21.

TABLE 20.—UNDEVELOPED POWER SITES AND POWER AVAILABLE ON SNAKE RIVER BETWEEN HUNTINGTON, ORE., AND LEWISTON, IDAHO.

Location.	Miles up stream from Lewiston.	Maximum Head.		With Existing Flow.				With Regulated Flow.			
				Discharge, in second-feet.		Horse-power, 0.08 H. Q.		Discharge, in second-feet.		Horse power, 0.08 H. Q.	
		90 per cent. of time.	50 per cent. of time.	90 per cent. of time.	50 per cent. of time.	90 per cent. of time.	50 per cent. of time.	90 per cent. of time.	50 per cent. of time.	90 per cent. of time.	50 per cent. of time.
Mile 154-158....	.....	90	70	8 000	16 500	57 600	92 500	6 900	11 200	49 700	62 800
Site No. 1.....	145.3	25	20	8 000	16 500	16 000	26 400	6 900	11 200	13 800	17 900
2.....	143.3	25	20	8 000	16 500	16 000	26 400	6 900	11 200	13 800	17 900
3.....	135.7	30	25	8 000	16 500	19 200	33 000	6 900	11 200	16 500	22 400
4.....	123.2	50	45	8 000	16 500	32 000	59 400	6 900	11 200	27 600	40 300
5.....	120.8	65	60	8 000	16 500	41 600	79 200	6 900	11 200	35 900	53 800
6.....	118.7	75	70	8 000	16 500	48 000	92 400	6 900	11 200	41 400	62 700
7.....	112.2	150	140	8 000	16 500	96 000	185 000	6 900	11 200	83 000	125 000
8.....	101.0	280	270	8 000	16 500	179 000	356 000	6 900	11 200	154 000	242 000
9.....	97.2	275	265	8 000	16 500	176 000	350 000	6 900	11 200	152 000	237 000
10.....	95.7	350	340	8 000	16 500	224 000	450 000	6 900	11 200	193 000	305 000
11.....	83.5	475	460	8 000	16 500	304 000	608 000	6 900	11 200	262 000	412 000
12.....	77.4	540	530	8 000	16 500	345 000	700 000	6 900	11 200	298 000	475 000
13.....	52.7	190	180	8 000	16 500	122 000	238 000	6 900	11 200	105 000	161 000
14.....	50.7	220	210	8 000	16 500	140 000	277 000	6 900	11 200	121 000	187 000
15.....	41.5	260	250	15 000	21 500	312 000	430 000	12 200	16 300	254 000	326 000
Salmon-Snake Tunnel Site....	77.4	530	530	3 800	5 100	161 000	216 000	3 800	5 100	161 000	216 000
Total.....	.....	...	...	.....	.....	919 000	1,510 000	.....	.....	800 000	1 130 000

NOTE:—Total includes only sites that will utilize full fall of river, namely, Mile 154-188, Site No. 1, and one-half of Sites Nos. 2, 3, 12, and 15, and Salmon-Snake Tunnel site.

TABLE 21.—UNDEVELOPED POWER SITES ON SALMON RIVER BETWEEN SALMON, IDAHO, AND ITS MOUTH WITH MINIMUM INTERFERENCE WITH RAILROAD LOCATION.

No.	Name.	Miles down stream from Salmon.	Head, in feet.	With Existing Flow.				With 800 000 Acre-Ft. Storage, Bruno Site.			
				Discharge, in second-feet.		Horse-power.		Discharge, in second-feet.		Horse-power.	
				90 per cent. of time.	50 per cent. of time.	90 per cent. of time.	50 per cent. of time.	90 per cent. of time.	50 per cent. of time.	90 per cent. of time.	50 per cent. of time.
12 JC 1	Mile 28	28.0	105	1 600	2 040	13 400	17 200	2 100	2 540	17 600	21 300
12 JC 2	Shoup	40.8	95	1 640	2 070	12 500	15 800	2 140	2 570	16 300	19 500
12 JC 3	Mile 45	45.0	80	1 670	2 110	10 700	13 500	2 170	2 610	13 900	16 700
12 JC 4	Long Tom	59.4	178	1 880	2 380	26 800	34 000	2 380	2 880	34 000	41 000
12 JE 1	Pinnacle	62.0	...	2 720	3 450	*	*	3 220	3 950	*	*
12 JE 2	Proctor Falls	65.4	78	2 720	3 450	17 000	21 500	3 220	3 950	20 100	24 700
12 JE 3	Horse Creek	73.4	105	2 780	3 520	23 400	29 600	3 280	4 020	27 600	33 800
12 JE 4	Mile 78.3	78.3	60	2 800	3 540	13 500	17 000	3 300	4 040	15 800	19 400
12 JE 5	Black Canyon	89.7	128	2 920	3 690	30 000	37 800	3 420	4 190	35 000	43 000
12 JE 6	Dillinger Creek	96.2	107	2 940	3 720	25 200	31 800	3 440	4 220	29 400	36 000
12 JE 7	Rattlesnake	99.6	40	2 950	3 740	9 450	12 000	3 450	4 240	11 000	13 600
12 JE 8	Growler	111.0	165	3 010	3 810	39 700	50 300	3 510	4 310	46 500	57 000
12 JE 9	Painted Rock	118.0	95	3 020	3 820	23 000	29 000	3 520	4 320	26 800	32 800
12 JG 1	Castle	127.6	105	3 500	4 420	29 400	37 000	4 000	4 920	33 400	41 000
12 JG 2	Crooked Bar	136.5	60	3 580	4 540	17 200	21 800	4 080	5 040	19 600	24 200
12 JG 3	Rheims	146.4	80	3 640	4 610	23 300	29 500	4 140	5 110	26 500	32 700
12 JG 4	Crevice	160.0	135	3 720	4 700	40 200	50 800	4 220	5 200	45 600	56 100
Total....		.....	1 616	.....	.....	354 750	448 600	.....	.....	419 100	512 800
		Below Riggins.									
12 JJ 1	Poodle Dog†	14.0	140	3 800	5 300	42 600	59 400	4 300	5 800	48 200	65 000
12 JJ 2	Rhett†	20.0	43	3 800	5 300	13 100	18 200	4 300	5 800	14 800	20 000
12 JJ 3	Red Canyon	41.7	190	3 800	5 300	57 800	80 600	4 300	5 800	65 500	88 200
12 JJ 4	Green Canyon	53.5	112	3 800	5 300	34 000	47 500	4 300	5 800	38 500	52 000
12 JJ 5	Snow Hole	63.5	95	3 900	5 500	29 600	41 800	4 400	6 000	33 400	45 500
12 JJ 6	Section Line	72.0	62	3 900	5 500	19 400	27 300	4 400	6 000	21 400	29 800
12 JJ 7	Lower Canyon	88.0	115	3 900	5 500	35 800	50 600	4 400	6 000	40 500	55 200
Total...		....	757	.....	.....	232 300	325 400	.....	.....	262 300	355 700
Grand Total		....	2 373	.....	.....	587 050	774 000	.....	....	681 400	868 500
Salmon-Snake Tunnel scheme ‡.		....	.....	.....	.....	173 000	242 000	.....	.....	196 000	265 000

* Site does not fit in with feasible scheme of development. † Used as power dams. ‡ Developed by means of Poodle Dog Diversion Dam into Snake; would destroy power value of Sites 12 JJ 1 to 12 JJ 7.

TABLE 22.—UNDEVELOPED WATER POWER SITES IN THE KLAMATH RIVER BASIN.

Index No.*	Stream utilized.	Head, in feet.	Plan 1.				Plan 2.				Plan 3.			
			Quantity, in second-feet.		Horse-power, 0.08 H. Q.		Quantity, in second-feet.		Horse-power, 0.08 H. Q.		Quantity, in second-feet.		Horse-power, 0.08 H. Q.	
			90 per cent. of time.	50 per cent. of time.	90 per cent. of time.	50 per cent. of time.	90 per cent. of time.	50 per cent. of time.	90 per cent. of time.	50 per cent. of time.	90 per cent. of time.	50 per cent. of time.	90 per cent. of time.	50 per cent. of time.
1	Williamson River	222	40	220	710	3 900	55	200	980	3 550	55	200	98	3 550
2	Spring Creek	15	200	350	240	420	200	350	240	420	200	350	240	420
3	Sycan River	300	15	42	360	1 000	15	42	360	1 000	15	42	360	1 000
4	Klamath River	270	343	1 240	1 400	26 800	980	980	21 200	21 200	0	0	0	0
5	"	700	343	1 240	19 200	69 500	980	980	55 000	55 000	0	0	0	0
6	"	400	343	1 240	11 000	39 700	980	980	31 400	31 400	0	0	0	0
7	"	125	343	1 240	3 400	12 400	980	980	9 800	9 800	0	0	0	0
8	"	180	400	1 300	5 800	18 700	1 040	1 040	15 000	15 000	60	60	860	860
9	Proposed Klamath-Shasta Canal	830	0	0	0	0	0	0	0	0	980	980	65 000	65 000
9	Proposed Klamath-Shasta Canal	800	0	0	0	0	0	0	0	0	0	980	0	62 700
9*	Shasta River	300	42	192	1 000	4 600	42	192	1 000	4 600	142	292	3 400	7 000
10	Klamath River	50	450	1 500	1 800	6 000	1 080	1 230	4 320	4 920	250	1 330	1 000	5 320
11	"	410	1 220	2 610	40 000	85 600	1 340	2 810	44 000	92 200	1 070	2 100	35 100	68 900
12	"	164	1 390	3 310	18 200	43 500	1 480	3 510	19 400	46 100	1 310	2 930	11 200	38 500
13	"	110	1 410	3 420	12 400	30 100	1 500	3 620	13 200	31 900	1 340	3 040	11 800	26 800
14	"	230	1 480	3 730	26 100	65 700	1 560	3 980	27 500	69 200	1 420	3 420	25 000	60 200
15	"	130	1 740	8 040	18 100	62 800	1 830	6 240	19 000	65 000	1 710	5 520	17 800	57 400
16†	"	171	1 810	6 400	24 800	87 600	1 900	6 600	26 000	90 400	1 790	5 870	24 500	80 400
17	"	83	1 820	6 440	12 100	42 700	1 910	6 640	12 700	44 100	1 800	5 920	12 000	39 300
18	"	43	2 390	10 400	8 220	35 800	2 970	10 000	10 200	34 400	2 870	9 340	9 880	32 100
19	South Fork Trinity River	90	26	145	187	1 040	26	145	187	1 040	26	145	187	1 040
20	Trinity River	190	.....	.....	.....	.....	800	800	12 200	12 200	800	800	12 200	12 200
21	"	270	191	1 360	4 130	29 400	878	1 270	19 000	27 500	878	1 270	19 000	27 500
22	"	220	235	1 720	4 140	30 300	925	1 480	16 300	26 000	925	1 480	16 300	26 000
23	"	220	468	2 800	8 240	49 300	1 080	2 260	19 000	39 800	1 080	2 260	19 000	39 800
24	"	120	496	2 850	4 760	27 400	1 120	2 380	10 800	22 900	1 120	2 380	10 800	22 900
Total		...	.....	.....	207 000	687 000	.....	.....	363 000	660 000	.....	.....	278 000	599 000

* Power Sites 1, 2, 3, 19, 20, 21, 22, 23, and 24 are not affected by development under Plans 1, 2, and 3.

† Alternate plan of development omitted in totals.

PLAN 1.—The values given under Plan 1 are based on the utilization of Upper Klamath Lake for storage to Elevation 4 143.3 above sea level, U. S. R. S. datum, with complete irrigation development in the Klamath River Basin above the California-Oregon boundary.

PLAN 2.—In Plan 2 it is assumed that Upper Klamath Lake will be utilized for storage to Elevation 4 158 above sea level, U. S. R. S. datum, and that water will be used for full irrigation development in the Klamath River Basin above the California-Oregon boundary.

PLAN 3.—This plan of development is the same as Plan 2, except that 980 sec-ft. is to be diverted from Klamath River at Keno, Ore., for the irrigation of 100 000 acres of land in Shasta Valley, California.

5.—At present there is no market for any considerable block of power and there is none in sight, a fact shown by the absence of applications for permits before the Federal Power Commission.

6.—The total undeveloped horse-power at 70% efficiency is as follows:

Without regulation of flow:

Available 90% of the time.....	387 000 h.p.
Available 50% of the time.....	774 000 h.p.

With regulation of flow:

Available 90% of the time.....	681 000 h.p.
Available 50% of the time.....	868 500 h.p.

Klamath River*

1.—Twenty-four sites on Klamath River and its principal tributaries have been examined.

2.—The greatest proportion of power will probably be developed by means of relatively low diversion dams from which water will be led in canals or conduits to points of use.

3.—The amount of power that can be developed will depend to a considerable extent on the quantity of water used for irrigation.

4.—The total undeveloped horse-power at 70% efficiency is as follows:

By Plan 1:

Available 90% of the time.....	207 000 h.p.
Available 50% of the time.....	687 000 h.p.

By Plan 2:

Available 90% of the time.....	363 000 h.p.
Available 50% of the time.....	660 000 h.p.

By Plan 3:

Available 90% of the time.....	278 000 h.p.
Available 50% of the time.....	599 000 h.p.

The estimates of power given in Tables 19, 20, 21, and 22, are based on the static head and over-all plant efficiency of 70 per cent.

Gerard H. Matthes,† M. Am. Soc. C. E. (by letter).—The discussions on this subject are illuminating, as they show the wide range of adaptability of aerial photography as a tool in the hands of the engineer; also by conveying some conception of the vast field of usefulness of this comparatively new art. To the Engineering Profession, the chief interest in aerial photography is now, and doubtless will continue to be, centered in devising simpler methods of constructing accurate maps, whether of the mosaic or of the hand-drawn type.

Col. Birdseye has set forth clearly the method of utilizing aerial photographs in the mapping operations of the U. S. Geological Survey, and it

* See Table 22.

† U. S. Asst. Engr., U. S. Engr. Office, Chattanooga, Tenn.

will be noted that these operations are quite different from those used by the writer. The reason for this is attributable to the fundamentally different principles which underlie the two systems of mapping. In the Geological Survey mapping, accuracy of geographical location is more important than accuracy of detail, whereas in the engineer's map, as exemplified by the survey of the Tennessee River described in the writer's paper, the opposite is true. In the Geological Survey method, it is not practicable to eliminate errors in scale or discrepancies caused by tilt or elevation displacements in individual photographs, because the plan of procedure supplies no information that will reveal the extent of such local errors and discrepancies. Reliance is placed, however, in a rigid ground control that maintains the accuracy of the completed map as regards principal dimensions, and the intervening detail is made to fit in by a process of adjustment that eliminates the errors in scale. The other discrepancies referred to, which are largely local in character, disappear or become negligible, due to the great reduction of the map in being brought to the scale of publication. In the engineer's map, however, such a procedure would not be admissible. The scale is large, and the photographs are taken as nearly as possible to the scale of the finished map. It is imperative, therefore, that each photograph be tested for errors, and corrected before being embodied in the map.

Mr. Hill's remarks refer entirely to maps of the mosaic type, that were supplemented by ample horizontal control derived from ground surveys. The writer has been impressed with the high degree of accuracy that was attained in the maps described by Mr. Hill. This accuracy was made possible through the use of improved methods of rectifying photographs, a process in which considerable progress has been made on the Pacific Coast, and which is responsible for the increasing use there of mosaic maps for engineering purposes. It is rapidly becoming recognized as the most efficient way of locating power transmission lines, highways, and railroads.

Mr. Tibbetts' method of taking oblique views is destined to become popular with engineers, as it requires no special training or equipment. The writer finds that with a rapid lens good work can be done, also with the camera pointed at right angles to the direction of flight. Excellent views have thus been obtained, from an altitude of about 100 ft., of objects about 500 ft. distant. The time of exposure was $\frac{1}{500}$ sec., and the speed of the aeroplane about 75 miles per hour. As stated by Mr. Tibbetts, plate cameras with focal plane shutters are best adapted to taking oblique views. As regards the indifferent results with ray filters reported by Mr. Tibbetts, it should be noted that haze caused by dust, forest fires, or soft coal smoke, acts as a screen that cannot be combated with filters; but haze in an undefiled atmosphere is capable of being clarified with astonishing results through the use of properly selected filters. Experience is required to decide what filter should be used under given conditions.

Mr. Herington's discussion of the advantages of aerial photography in the mapping of timbered areas is most timely. There is probably no more exasperating undertaking for an engineer or surveyor than the preparation

of a reliable map of a densely wooded region by the older survey methods. The physical obstacles are usually such as to render only an inferior degree of accuracy obtainable, and that at considerable expenditure of time and money. A good photographic mosaic on an adequate scale offers the most elegant means of obtaining an accurate base map of heavily timbered country for any purpose, especially where the land is sectionized and care has been taken to mark the section corners so as to cause them to show plainly in the photographs. Mr. Herington's estimate of 15 to 20 cents per acre for aerial photographic work would make available a first-class mosaic on a scale of 400 ft. to the inch, a scale which would be well adapted for timber valuation work, and would bring out a wealth of information not otherwise obtainable.

Only one aerial timber survey of a large tract has come to the writer's notice. This was made in Canada for a paper manufacturing company, and is of interest in that the photographs made it possible to identify readily the several species of trees.

The writer's experience, in mapping backwoods sections in Tennessee and Alabama, has convinced him that aerial photographs offer by far the most reliable means of mapping extensive timbered areas yet devised. It is not until the stereoscopic properties of the photographs are brought into play, however, that their maximum value can be realized. This brings up the subject on which Mr. Herington requests additional information, namely, the uses of the stereoscope.

A certain amount of skill has to be acquired through practice in order to use the stereoscope effectively. This skill consists in training the muscles of the eye and in learning to interpret the full meaning of what is seen. It must be remembered that through the stereoscope one is enabled to visualize the face of the earth in a manner that is without precedent to the uninitiated. Here, then, is a new aspect of things that must be studied in order to yield results. Even the aviator, from his lofty station, can form no conception of what the stereoscope may bring out, for his eye sees only a mere fraction of the detail recorded by a modern camera lens. To the man in the air, the world seems flattened to an absurd degree, and all details appear insignificantly small. The stereoscope, on the other hand, not only restores the relief with surprising fidelity, but brings to the attention an astonishing amount of interesting detail that otherwise would remain meaningless or unnoticeable. For instance, a small path or a logging road through a timber tract, which would escape detection in an ordinary photographic print, in the stereoscope stands out clearly under the trees, and the latter assume an individuality that differentiates them from their shadows on the ground. To the naked eye, the blur caused by deciduous trees without leaves and their own shadows is practically meaningless, but in the stereoscope there is no difficulty in singling out each tree and each shadow. This statement cannot be realized until tested out by actual experience. Again, a fine line appearing in a photograph may be a ditch, a fence, or the shadow of an embankment, and, from a single photograph, it may be quite impracticable to decide which it is. The stereoscope will remove all doubts. Much that is obscure or unintelligible in the

average photograph becomes clear and obvious when viewed in the stereoscope. Heretofore, the possibilities of this instrument in connection with aerial photographic mapping have scarcely been realized. The writer has reason to believe that its importance will grow as its uses become better understood. Of late, its value in expediting the mechanical work involved in the Bagley method of radial control, described in the writer's paper, has become manifest. For instance, in selecting control points common to two or more photographs, considerable difficulty was often found in determining the identical location of a point on the several prints, a requirement which is of prime importance in order to secure accuracy. This arose especially where, as in a forested region, there was uniformity of detail that offered no salient features or landmarks. In such cases, it has been found that the location of the point can be quickly duplicated by viewing the prints in the stereoscope and simply using a fine-pointed tool to mark the desired point at the place seen stereoscopically. Surprisingly quick and accurate results can be thus obtained. In the survey of the Tennessee River, the stereoscope was used extensively in both field and office in examining terrain in which the monotony, as viewed from a single photograph, was baffling. In a forest of deciduous trees, for instance, the unbroken array of tree tops rarely furnished any clue as to the topographic features underneath, until viewed in the stereoscope. Then, ridges, gulches, slopes, and plateaus, the existence of which could not be suspected, would at once come into plain view. Similarly, cultivated fields of distressingly uniform appearance would, in the stereoscope, unfold all the minor rises, benches, and drainage courses that usually characterize them in Nature, but which are rarely visible in a vertical photograph.

The system of photographic mapping adopted in the survey of the Tennessee River—which formed the main subject of the writer's paper—is being further developed. Congress has ordered the survey to be resumed, and has appropriated the sum of $200 000 for the purpose. In Alabama, the Alabama Power Company has adopted the same system in mapping about 500 sq. miles of the Tallapoosa River Valley for the purpose of planning a series of hydro-electric projects. The writer is confident that the continued use of this new system of mapping will operate to develop and improve it until its efficiency and accuracy will be such as to make it one of the most attractive methods of making maps for engineering purposes.

Frederick Hall Fowler,* Assoc. M. Am. Soc. C. E. (by letter).—Since the preparation of the writer's paper, several important reports on the power resources of the West have been published, or made available for public inspection. Among these are the "Report to the Federal Power Commission on the Uses of the Upper Columbia River", and three of the four reports mentioned by Mr. Hoyt. The fourth report ("Klamath River and Its Utilization", by E. C. La Rue, M. Am. Soc. C. E.) was previously open to inspection and was compared with certain other information by the writer before preparing the map, Fig. 28.

* Civ. Engr., San Francisco, Calif.

The report on "The Uses of the Upper Columbia River", published by the Federal Power Commission, will be of great interest to students of Western water power as it contains the detailed results of study by a special Board of Engineers.

The general results of the four reports prepared by the U. S. Geological Survey are stated and are shown in tabular form by Mr. Hoyt. His Tables 19 to 22, inclusive, are in far greater detail than was attempted by the writer, but are in accord with his more general statements. They show, as might be anticipated, larger power resources than were recorded on Fig. 28.

It must be borne in mind that all studies of large power streams as yet practically untouched will be at best only approximations of the ultimate program, and the wide variations in rate of development, or in the balance between power and irrigation, may be caused by conditions that cannot now be foreseen. The very interesting detailed information given in Mr. Hoyt's discussion only goes to prove even more emphatically the super-abundance of power in the Pacific Northwest.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 1522

THIRD PROGRESS REPORT OF THE SPECIAL COMMITTEE TO REPORT ON STRESSES IN RAILROAD TRACK*

WITH DISCUSSION BY S. TIMOSHENKO, ESQ.

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November 29, 1922

* Presented to the Annual Meeting, January 17, 1923.
† Mr. Hunt died on July 11, 1923.

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To the American Society of Civil Engineers:

The Special Committee to Report on Stresses in Railroad Track herewith presents its third progress report.

I.—Introduction

1.—Preliminary.—Since its organization in 1914, the Committee has been co-operating with the Special Committee on Stresses in Railroad Track appointed by the American Railway Engineering Association, the membership of the two committees being the same with the exception of one member of the American Railway Engineering Association who does not hold membership in the American Society of Civil Engineers, and the work has been carried on as that of one committee. This report is presented simultaneously to the two Societies and to the American Railway Association, which also has been co-operating in the work for the past four years.

As stated in the earlier reports, the Committee has felt that an adequate report on stresses in railroad track must be based largely on experimental data derived from extensive tests on standard railroad track, and that, in view of the complexity of the action of track under load and the variability of the conditions to be found in track and load, the work of conducting experiments and reducing the data would necessarily require much time and effort. The development of methods of conducting the tests and the work of devising the instruments and apparatus have involved a considerable expenditure of time, effort, and money. It has been recognized that in obtaining data on the action of track under variable conditions of both track and load, great refinement of method was not possible and that it was important to make tests under conditions of railroad service as nearly normal as possible, also utilizing, of course, the data of laboratory investigations where conditions would not warrant satisfactory experiments in the field.

The first progress report of the Committee was published in Vol. LXXXII of the *Transactions* of the Society (1918). It may also be found in Vol. 19 of the *Proceedings* of the American Railway Engineering Association (1918). In this report, a method of analysis for determining the moments and stresses in the rail by considering the track as an elastic structure was presented, the instruments and methods of testing were described, the depression of the track and the rail profile under load were given, and the stresses developed in the rail under a variety of loads were recorded. The report relates principally to stresses in rail and to the general elastic conditions of the track.

The second progress report of the Committee was published by the Society in Vol. LXXXIII of *Transactions* (1919-20) and in *Proceedings* for February, 1920. It may also be found in Vol. 21 of the *Proceedings* of the American Railway Engineering Association, in *Bulletin 224* of the Association, and in *Circular No. S-II-10* of the American Railway Association. This report gives the results of tests on two railroads, with several types of locomotives, to determine the stresses in rail in relation to speed and counterbalance effect.

It records the results of tests on two railroads to determine the depression of track and the flexure of ties and their action under load for a variety of conditions found in track. Tests to find the manner and the principles involved in the transmission of pressure from one or more ties downward and laterally through ballast material were reported and an analytical consideration of the transmission of pressure was given.

The work reported herein includes tests on straight track and curved track. In the tests on straight track, consideration was given to the effect of counterbalance, the effect of speed and the combined effect of speed and counterbalance, and the lateral bending moments and stresses in the rail. In the tests on curved track are taken up the general action of curved track as the locomotive traverses the curve, the magnitude of the vertical bending stresses in the rail and the corresponding vertical loads producing them as contrasted with the normal loads, the lateral bending moments and stresses under the several wheels, the distortion of the alignment of the curves, and the general effect of speed, degree of curve, and super-elevation. The tests were conducted on the Illinois Central Railroad, the Delaware, Lackawanna and Western Railroad, the Atchison, Topeka and Santa Fe Railway, and the Southern Pacific Railroad. In the tests, thirteen locomotives of eight distinct types were used. In several types, there were variations in the design of the locomotives of the different railroads. The report is presented under the heads of Tests on Straight Track and Tests on Curved Track.

The Committee is continuing work on the subject assigned to it.

2.—Acknowledgment.—Since the second progress report was issued, the funds for use in carrying on the work of the Committee have been taken principally from contributions made by the American Railway Association. The Committee expresses its appreciation of this support. Expenditures have also been made from funds in the hands of the American Society of Civil Engineers and the American Railway Engineering Association contributed by the United States Steel Corporation, the Bethlehem Steel Company, the Lackawanna Steel Company, and the Cambria Steel Company.

The co-operation of railroad companies in furnishing facilities for the test work has itself been a large contribution. The Illinois Central Railroad the late A. S. Baldwin, M. Am. Soc. C. E., Vice-President, and F. L. Thompson, Chief Engineer; the Delaware, Lackawanna and Western Railroad, G. J. Ray, M. Am. Soc. C. E., Chief Engineer, and A. J. Neafie, Principal Assistant Engineer; and the Atchison, Topeka and Santa Fe Railway, C. F. W. Felt, M. Am. Soc. C. E., Chief Engineer, have been liberal in providing locomotives, track, and other facilities in the conduct of the tests. The Atchison, Topeka and Santa Fe Railway and the Southern Pacific Company have also made it possible for the Committee to use such portions of the results of the tests made on the lines in California as are related to the subject matter of this report. A number of engineers from the Engineering, Mechanical, and Test Departments of the Atchison, Topeka and Santa Fe Railway gave helpful assistance throughout the work. The services of Mr. C. C. Huycke in aiding in the reduction of the data were also contributed.

Mr. E. E. Cress, Assistant Engineer of Tests, in charge of field and office work and the reduction of data and preparation of material, is entitled to much credit; his thorough familiarity with details, keen grasp of the problem, and helpfulness in the study and preparation of the data have made his services particularly valuable. Louis J. Larson, Jun. Am. Soc. C. E., Associate in Theoretical and Applied Mechanics in the University of Illinois, by reason of his special training and insight in testing work, has given valuable service in both laboratory and field work during the summer seasons. Messrs. H. L. Parr and R. Ferguson have given helpful service in the reduction of data and the preparation of the report. Others have assisted in the work from time to time and all have given loyal and careful service.

The University of Illinois has continued to co-operate in the work by giving the use of laboratory, shop, and office facilities, and through the service of members of the staff of the Engineering Experiment Station from time to time.

The Committee records with profound regret the loss of a member by death, Mr. Archibald Stuart Baldwin, who from the time of the organization of the Committee took great interest in the work and was active in the making of plans, the provision of means for carrying on the tests, and the discussion and interpretation of the results. He was especially helpful in making the arrangements for the extended use of locomotives and track of the Illinois Central Railroad for test work.

TABLE 1.—PROPERTIES OF SECTIONS OF RAILS USED IN THE TESTS.

Rail section (full section).	Area, in square inches.	Moment of Inertia.		Section Modulus.		
		For horizontal axis.	For vertical axis.	Horizontal axis.		Vertical axis.
				Base.	Head.	Base.
85-lb. Am. Soc. C. E.	8.3	30.1	....	12.2	11.1	....
90-lb. A. R. A.-A.	8.8	38.7	7.5	15.2	12.6	2.9
90-lb. S. F.						
92-lb. frictionless	9.0	32.0	8.3	14.7	10.1	3.1
101-lb. D. L. & W.	9.9	38.9	9.5	15.8	13.1	3.5
105-lb. D. L. & W.	10.2	50.2	9.5	18.0	15.7	3.5

3.—The Track.—The tests on the Atchison, Topeka and Santa Fe Railway, made in September and October, 1920, were conducted on a section of single track of the 10° curve at the west approach to the bridge over the Mississippi River, at Fort Madison, Iowa, and on the east-bound tangent track in Illinois about 2 miles east of this point; also on tangent track and curves (single track) near Ribera, N. Mex., about 27 miles west of Las Vegas. The location, weight of rail and its date of laying, tie-spacing, depth of ballast, grade, gauge, super-elevation of outer rail, and corresponding speed are given in Table 2. The ties were 6 by 8 in. by 8 ft., mostly oak in the test track in New Mexico and Iowa, and pine in Illinois. Tie-plates were in use on all ties. No curve braces were in use. The ballast in the track in New Mexico was rock

and that in Iowa and Illinois was gravel. The test section of the 10° curve at Ribera was on a fill of 12 to 14 ft.; the 10° curve at Fort Madison was on 25-ft. fill. The 6° curve at Ribera was in a shallow cut and the tangent on ground nearly at grade. The tangent in Illinois was on 6-ft. fill. The rails were laid with alternate joints. The track was used as it was found. The 10° curve at Fort Madison had recently been re-surfaced and re-aligned and was in excellent condition. The other track had not been tamped recently, but it was in fair surface and alignment.

TABLE 2.—DATA OF THE TRACK.

Tie-spacing, depth of ballast, and super-elevation are given in inches, and gauge, in feet and inches.

Track.	Location.	Rail section.	Date of laying.	Tie-spacing.	Depth of ballast.	Grade, percentage.	Gauge.	Super-elevation of outer rail.	Speed of super-elevation, in miles per hour.
Tangent.....	Ribera, N. Mex.....	90-lb. S. F....	1917	18	12	—1.40	4-8½	.....	.....
6° curve.....	" "	"	1917	18	10	—1.18	4-8⅝	4.7	35
10° curve....	" "	"	1920	19	10	—1.02	4-8⅞	4.7	27
Tangent	East Fort Madison, Ill.	85-lb. Am. Soc. C. E.	1906	20	22	0.0	4-8⅝	.....	.....
10° curve....	Fort Madison, Iowa...	90-lb. S. F....	1920	21	14	0.0	4-9⅛	2.0	17
Tangent.....	Dover, N. J.....	105-lb. D. L. & W.....	1915	22	12	—0.55	4-8¾	.....	.....
4° curve.....	Dover, N. J.....	105-lb. D. L. & W.....	1919	22	18	—0.63	4-8⅞	3.7	37
6° curve.....	Mount Tabor, N. J..	92-lb. Frictionless.....	1914	22	6	—0.84	4-8¾	8.5	46
		101-lb. D. L. & W.....	1915						
7½° curve....	Paterson, N. J.....	105-lb. D. L. & W.....	1919	20	9	0.0	4-9	6.4	36
10° curve....	Bealville, Calif.....	90-lb. A. R. A.-A	1921	22	5 on old grade	2.00	4-8⅝	4.4	26
10° curve....	Cajon, Calif.....	90-lb. S. F.....	1920	18	10	1.80	4-8½	5.4	29
7° curve.....	Murphysboro, Ill.....	90-lb. A. R. A.-A	1914 1919	20	12	0.0	4-9⅛	3.7	28
14° curve ...	Champaign, Ill.....	85-lb	1903	19	Cinder	0.0	4-8⅞	3.8	20

The tests on the Delaware, Lackawanna and Western Railroad, made in July and August, 1920, were conducted on east-bound tangent track at Dover, N. J., on east-bound 4° curve east of Dover, on east-bound 6° curve east of Mount Tabor, N. J., and on east-bound 7½° curve at Paterson, N. J. Information about the track is given in Table 2. The ties were 7 by 9 in. by 8 ft. 6 in.; creosoted oak ties, creosoted oak and pine ties, and creosoted pine ties, respectively, were used on the three test sections. Tie-plates were used throughout and screw spikes were in use; the 7½° curve also had some driven spikes. The ballast was rock, rock on cinder, rock on gravel, and rock on sand. The tangent was a light fill, the 4° curve nearly at grade, the 6° curve on 10-ft. fill, and the 7½° curve in 5-ft. cut. The rails were laid with alternate joints. The track was tested as found and was in good surface and alignment.

The tests on the Illinois Central Railroad, made in June and July, 1920, were conducted on a 7° curve at Murphysboro, Ill., and on a 14° curve at Cham-

paign, Ill. The track was tested as found; the surface and alignment were in fairly good condition. The 7° curve had rock screenings for ballast. The 14° curve had cinder ballast. The gauge of the 7° curve was 4 ft. 9⅛ in., and that of the 14° curve, 4 ft. 8⅞ in. Tie-plates were in use.

The track at Bealville and Cajon, Calif., used in tests in 1922, is described in the discussion of the tests at these two locations.

From Table 2 it is seen that there are some differences in the gauge of the track. For example, on straight track the gauge on the test section of the Atchison, Topeka and Santa Fe Railway, at Ribera, was 4 ft. 8½ in., at East Fort Madison, it was 4 ft. 8⅝ in., and on the Delaware, Lackawanna and Western Railroad, at Dover, 4 ft. 8¾ in. The gauge of the 6° curve of the Delaware, Lackawanna and Western Railroad, at Mount Tabor, was 4 ft. 8¾ in., and that of the Atchison, Topeka and Santa Fe Railway, at Ribera, 4 ft. 8⅞ in. The gauge of the 10° curve at Fort Madison was 4 ft. 9⅛ in., at Bealville, 4 ft. 8⅞ in., and the track at Cajon was spiked to standard gauge of 4 ft. 8½ in., at the time of the test. The foregoing values were obtained by averaging several measurements. A variation of as much as ¼ in. may be found in a distance of 5 or 10 ft. It is to be expected that the gauge will widen with use after re-alignment, although measurements at given points showed no change after 4 or 5 days of testing.

The recommendation of the American Railway Engineering Association, as given in the "Manual", is to use standard gauge (4 ft. 8½ in.) for straight track and for curves up to and including 8°. It is also recommended that the gauge be widened ⅛ in. for each 2° or fraction thereof over 8° to a maximum of 4 ft. 9¼ in. It is stated that under ordinary conditions it is not necessary to re-gauge track if the increase in gauge has not amounted to more than ½ in., providing such increase is uniform, but the gauge, including widening due to wear, should never exceed 4 ft. 9⅛ in. The foregoing recommendation would make the gauge of a 10° curve 4 ft. 8⅝ in., with an allowable increase of ½ in. before re-alignment.

4.—The Locomotives.—Figs. 1 and 2 show the diagrams of the locomotives used on the tests of track on the Atchison, Topeka and Santa Fe Railway in New Mexico, Iowa, and Illinois, and Fig. 3, those used on track of the Delaware, Lackawanna and Western Railroad in New Jersey; wheel loads and spacings, diameter of drivers, crank-pin radius, and the number and type of the locomotive are noted. Fig. 4 shows a diagrammatic representation of the equalizing system of six of the locomotives. Tables 3 to 7, inclusive, give data of the counterbalancing of the drivers of the locomotives of the Atchison, Topeka and Santa Fe Railway, as furnished by the Mechanical Department of that Company, and Tables 8 and 9 give similar data furnished by the Delaware, Lackawanna and Western Railroad. The footnotes to these tables give the approximate modification in the counterbalance obtained by the method outlined in the second progress report for finding the effect of the outside rotating parts not being in the plane traversed by the counterweights.

The locomotives were used as they were found in service; all were in ordinary working order. Contours of the tires of the driving wheels were

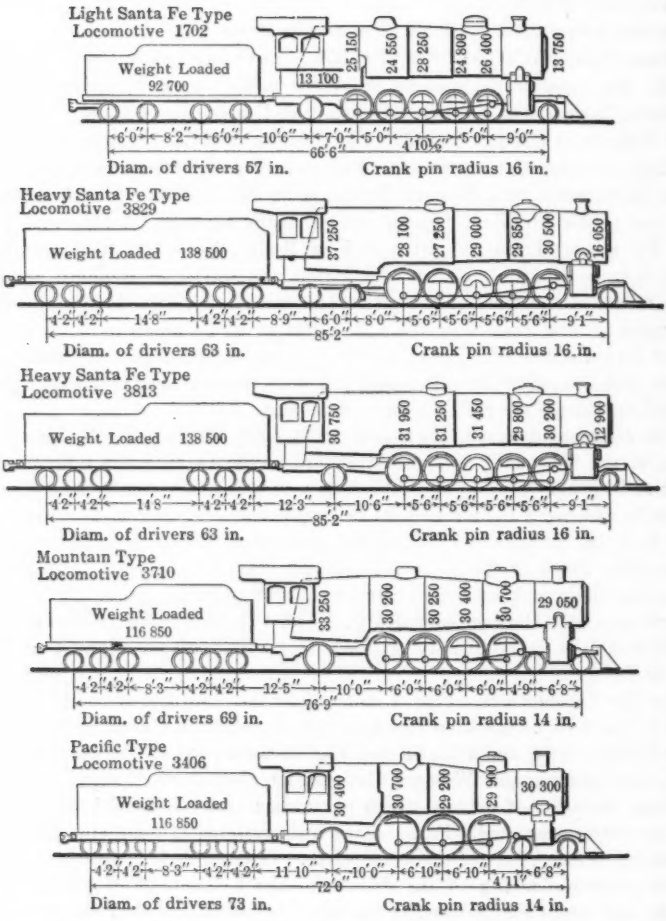
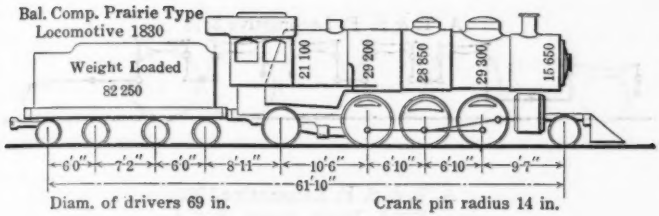


FIG. 1.—DIAGRAMS OF LOCOMOTIVES OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.



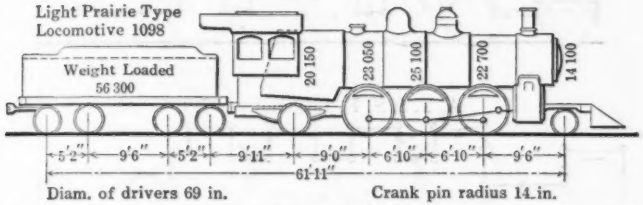
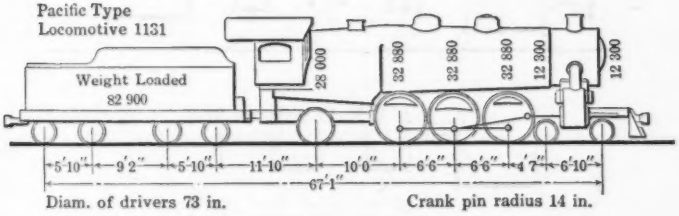


FIG. 2.—DIAGRAMS OF LOCOMOTIVES OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.



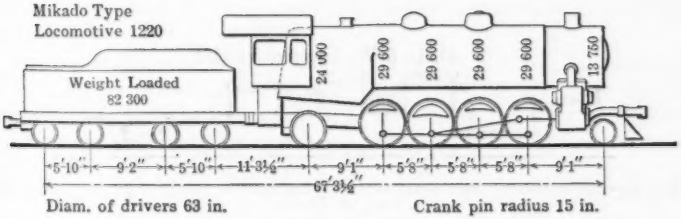


FIG. 3.—DIAGRAMS OF LOCOMOTIVES OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

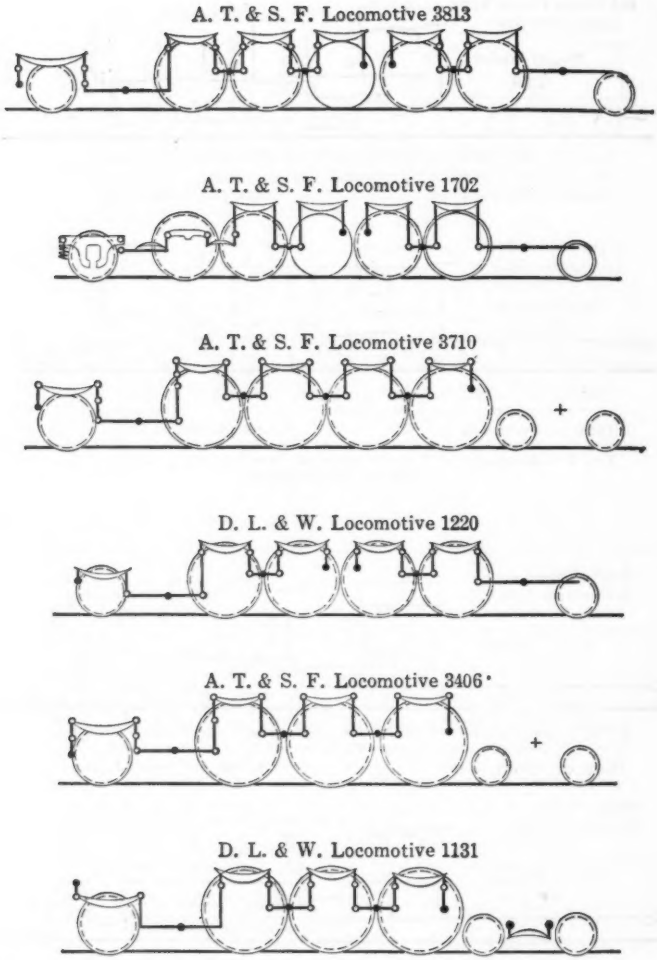


FIG. 4.—DIAGRAMMATIC REPRESENTATION OF EQUALIZING SYSTEMS OF LOCOMOTIVES USED IN TESTS.

taken; the tires may be said to have been in average condition—neither newly turned nor badly worn. No measurement of the lateral play of the driving axles in their boxes was made. The practice of the Atchison, Topeka and Santa Fe Railway is to provide a total lateral play of $\frac{1}{8}$ to $\frac{5}{16}$ in. at the boxes of the drivers when the locomotive leaves the shop. The regulations of the Interstate Commerce Commission provide that this lateral play shall not be permitted to exceed $\frac{3}{4}$ in.

TABLE 3.—Weights of Rotating and Reciprocating Parts, in Pounds, Pacific Type Locomotive of the Atchison, Topeka and Santa Fe Railway.

Reciprocating Parts.

Piston	706
Cross-head	606
Union link and lower end of combination lever	60
40% main rod, weight on cross-head pin	433
	1 805

$$\frac{0.50 \times 1\,805}{3} = 301$$

Rotating Parts.

Driver number.	3	Main.	1
50% of weight of reciprocating parts	301	301	301
Main rod, weight on crank-pin	....	650	....
Side rod, weight on crank-pin	190	705	190
Crank-pin	117	367	117
Crank-pin hub	89	153	89
One-half of eccentric crank	....	82	....
Total	697	2 258	697
Equivalent weight required at center of counterweight	372	1 435	372
Weight used	372	1 435	372
Difference in weight required and weight obtained	0	0	0
Equivalent difference at crank-pin center	0	0	0
Overbalance or underbalance at crank-pin circle for rotating parts only	+301	+301*	+301

* If the effect of the rods and pin not being in the plane of the counterweight is calculated by the method given in the second progress report, this becomes approximately an overbalance of 35 lb.

The Santa Fe type locomotive received the name from its introduction by the Atchison, Topeka and Santa Fe Railway. The 1674 class, the first of this type to be built by any railroad, is here called the light Santa Fe type. No. 1702, the one used in the test, was built in 1903. The small size of the drivers made full counterbalancing on the main driver impracticable, and counterbalancing devices, known as bobs, were attached to the driver axle inside the bearings. As the driver loads and spacing of these locomotives are relatively small, the stresses developed in the rails used were smaller than those found with the other locomotives, even though the effect of speed, counterbalance, and lateral bending was considerable. They were built, of course, for use on lighter rail sections. It should be noted that many Santa Fe type loco-

motives with 57-in. drivers still remain in use on a number of railroads of the United States, and some of these locomotives are not of as good design as the 1674 class locomotives. The Heavy Santa Fe type locomotive was not only much heavier, but the design was greatly improved, and its general performance on the track was much better than the lighter locomotive; No. 3813, the one used, had been in service about a year, as had the Mountain type and the Pacific type of the Atchison, Topeka and Santa Fe Railway. Locomotives of the Prairie type used had been in service from 15 to 20 years. The only change made in the locomotives during the tests was in the Mountain type; the lateral spring holding the trailer was removed for the last part of the tests. The Double Trailer Heavy Santa Fe type is the term given to the one locomotive which the railroad had equipped with four trailing wheels for experimental purposes; this is the only locomotive to which the double trailer had been applied.

TABLE 4.—WEIGHTS OF ROTATING AND RECIPROCATING PARTS, IN POUNDS, LIGHT PRAIRIE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

RECIPROCATING PARTS.

Piston	752
Cross-head	392
Weight of main rod on cross-head pin	297
	1 441

$$\frac{0.666 \times 1\,441}{3} = 320$$

ROTATING PARTS.

Driver number.	3	Main.	1
Two-thirds of weight of reciprocating parts	320	320	320
Main rod, weight on crank-pin	...	297	...
Side rod, weight on crank-pin	163	1 100	170
Crank-pin	100	280	100
Crank-pin hub	90	160	90
Total	673	2 157	680
Equivalent weight required at center of counterweight	410	1 630	414
Weight used	410	1 630	414
Difference between weight required and weight obtained	0	0	0
Equivalent difference at crank-pin circle	0	0	0
Overbalance or underbalance at crank-pin circle for rotating parts only	+320	+320	+320

The Mikado type locomotive of the Delaware, Lackawanna and Western Railroad was built in 1913 and the Pacific type used was built in 1915. These two types of locomotives were also used in the tests made on this railroad in 1916.

In the tests made in California, the locomotive of the Atchison, Topeka and Santa Fe Railway used was the same as the Heavy Santa Fe type shown in Fig. 1, except that on account of the use of oil burners the weight on the

trailer was only 22 900 lb. That of the Southern Pacific Company had nearly the same driver loads, but the load on the trailer was 29 400 lb. The Mikado type locomotive used in the preliminary tests on the Illinois Central Railroad was of the same class as that used on that railroad in 1918. The Ten-Wheel type was a much lighter locomotive.

TABLE 5.—WEIGHTS OF ROTATING AND RECIPROCATING PARTS, IN POUNDS, MOUNTAIN TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

RECIPROCATING PARTS.

Piston	904
Cross-head	610
Union link and lower end of combination lever	62
40% main rod, weight on cross-head pin	522
	2 098

$$\frac{0.50 \times 2\ 098}{4} = 262$$

ROTATING PARTS.

Driver number.	4	3	Main.	1
50% of weight of reciprocating parts	262	262	262	262
Main rod, weight on crank-pin	....	....	783	....
Side rod, weight on crank-pin	180	530	665	175
Crank-pin	135	130	430	100
Crank-pin hub	129	129	266	129
One-half eccentric crank	....	....	87	....
Total	706	1 051	2 493	666
Equivalent weight required at center of counterweight	417	640	1 939	417
Weight used	417	640	1 939	417
Difference between weight required and weight obtained	0	0	0	0
Equivalent difference at crank-pin circle	0	0	0	0
Overbalance or underbalance at crank-pin circle for rotating parts only	+262	+262	+262*	+262

*If the effect of the rods and pin not being in the plane of the counterweight is calculated by the method given in the second progress report, this becomes approximately an underbalance of 35 lb.

5.—The Conduct of the Tests and the Reduction of the Data.—The methods used in the tests of track were the same as those used in the tests described in the first and second progress reports. Eight stremmatographs were used simultaneously. (A description of these instruments will be found in the first progress report*). Four stremmatographs were placed on one rail between ties at distances apart approximately equal to the average spacing of the drivers of the locomotive used on the test, and the other four instruments directly opposite on the other rail. The driving mechanism used rotated the disks of all the instruments simultaneously. The correlation of a point on the record of one of the disks with the point of any other disk at the same moment was possible. As the locomotive passed the test section a record of the strains in the rail was made on each instrument. The passage of each wheel of one side

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1224.

of the locomotive and tender thus was recorded on four instruments and each wheel of the other side on four other instruments. A run then gave records of what happened under one pair of wheels on eight instruments, and as each instrument holds two disks, sixteen records in all were made. As usually operated, a disk would hold the records of four runs. Fresh disks would then be inserted.

TABLE 6.—WEIGHTS OF ROTATING AND RECIPROCATING PARTS, IN POUNDS, LIGHT SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

RECIPROCATING PARTS.

Piston	985
Cross-head	480
Union link and lower end of combination lever	65
48% main rod, weight on cross-head pin	470
	2 000

$$\frac{0.666 \times 2\,000}{5} = 267$$

ROTATING PARTS.

Driver number.	5	4	Main.	2	1
Two-thirds of weight of reciprocating parts	267	267	267	267	267
Main rod, weight on crank-pin	....	....	510	....	....
Side rod, weight on crank-pin	125	302	762	314	125
Crank-pin	70	70	390	70	67
Crank-pin hub	157	157	325	157	157
Total	619	796	2 254	808	616
Equivalent weight required at center of counterweight	604	836	2 365	848	602
Weight used	588	760	1 495	802	579
Difference between weight required and weight obtained	−16	−76	−870	−46	−23
Equivalent difference at crank-pin circle	−17	−72	−828	−44	−23
Overbalance or underbalance at crank-pin circle for rotating parts only	+250	+195	−561*	+223	+244

* No information is at hand on the weight or position of the bobs or position of planes of the outside rotating parts.

The use of eight stremmatographs—four had been the largest number used in previous tests—had advantages even on the tests of straight track. The strain under companion drivers was recorded simultaneously; in fact, with just the right spacing, the strain under four pairs of wheels was recorded at the same instant, thus giving the instantaneous effect of the four pairs of drivers. The distance from the first instrument to the fourth was generally about equal to the circumference of a driver; this facilitated finding the effect of counterbalance. The use of eight instruments was more likely to give the general effect of the locomotive as a whole, at a given time, than the smaller number. The time taken to obtain an adequate number of data was shortened, thus reducing the time the locomotive was taken from service.

The chief purpose in using eight instruments, however, was to facilitate the tests on curved track, where, of course, the stresses in the two rails are

generally unequal. The use of two disks in each stremmatograph, one recording the strain at one edge of the base of rail and the other that at the other edge, permitted the measurement of the lateral bending strains in the rails of curved track. It is believed that these measurements of the bending stresses developed in curved track were the first to be made in any tests. The means used were entirely successful. The preliminary tests made to learn the precautions and methods necessary for the satisfactory conduct in the testing of curved track were made on a curve of a branch line. It will be noted that by the method used the stresses in the two edges of both inner and outer rail of the curved track, caused by both the vertical bending and the lateral bending of the rail, were measured under four pairs of drivers at about the same time.

TABLE 7.—WEIGHTS OF ROTATING AND RECIPROCATING PARTS, IN POUNDS, HEAVY SANTA FE TYPE LOCOMOTIVES OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

RECIPROCATING PARTS.

Piston	1 070
Cross-head	705
Union link and lower end of combination lever	62
40% main rod, weight on cross-head pin	585
	2 422

$$\frac{0.50 \times 2\,422}{4} = 303^{*}$$

ROTATING PARTS.

Driver number.	5	4	Main.	2	1
50% of weight of reciprocating parts	308	308	....	308	308
Main rod, weight on crank-pin	....	....	877	....	....
Side rod, weight on crank-pin	172	493	785	454	166
Crank-pin	118	130	500	130	105
Crank-pin hub	144	144	271	144	144
One-half eccentric crank	....	....	91	....	....
Total	742	1 075	2 524	1 036	723
Equivalent weight required at center of counterweight	574	899	2 714	866	564
Weight used	574	899	2 693	866	564
Difference between weight required and weight obtained	0	0	21	0	0
Equivalent difference at crank-pin circle	0	0	20	0	0
Overbalance or underbalance at crank-pin circle for rotating parts only	+308	+308	−20†	+308	+308

*It was not possible to counterbalance for the whole of the rotating weight in the main wheel; the deficiency of 21 lb. was distributed equally among the other drivers, bringing their total up to 308 lb.

†If the effect of the rods and pin not being in the plane of the counterweight is calculated by the method given in the second progress report, this becomes approximately an underbalance of 360 lb.

Unless otherwise noted, in all runs steam was shut off as the locomotive approached the test section of track, and the locomotive "coasted" past the instruments. The speeds were read from a speedometer in the cab connected with the tread of the trailer or the front wheel of the tender, the instrument having been checked up over a measured length of track. The locomotive was

then backed over the track and the next run made. For each set of tests, the order of speeds for consecutive runs was the same, varying from the lowest speed to the highest.

TABLE 8.—Weights of Rotating and Reciprocating Parts, in Pounds, Pacific Type Locomotive of the Delaware, Lackawanna and Western Railroad.

Reciprocating Parts.

Piston	961
Cross-head	585
Weight of main rod on cross-head pin	335
	1 881

$$\frac{2 \times 1\,881}{3 \times 3} = 418$$

Rotating Parts.

Driver number.	3	Main.	1
Two-thirds of weight of reciprocating parts	418	418	418
Main rod, weight on crank-pin	....	627	....
Side rod, weight on crank-pin	159	517	145
Crank-pin	116	398	114
Crank-pin hub	113	226	113
One-half eccentric crank	....	60	....
Total	806	2 246	790
Equivalent weight required at center of counterweight	478	1 562	468
Weight used	478	1 562	468
Difference between weight required and weight obtained	0	0	0
Equivalent difference at crank-pin circle	0	0	0
Overbalance or underbalance at crank-pin circle for rotating parts only	+418	+418*	+418

*If the effect of the rods and pin not being in the plane of the counterweight is calculated by the method given in the second progress report, this becomes approximately an overbalance of 175 lb.

The position of the counterweight with respect to one instrument was observed for each run. It was found unnecessary to control the position of counterweight on different runs; the starting and stopping and the running over curves within the length of the runs gave sufficient distribution of the position of counterweight with respect to the instruments throughout the revolution of the driver.

Fig. 5 shows the position of the stremmatographs in one of the tests of curved track. Variations from this arrangement were made, but the diagram is representative of all the tests. As all track was laid with alternate joints, it was necessary that a rail joint lie within the test section. Generally, the joint was between the two middle instruments, sometimes on one rail and sometimes on the other. A study of the records of the several instruments failed to show any general characteristics of the strains recorded in the instrument nearest the rail joint.

The process followed in the reduction of the data obtained by the stremmatographs was the same as that which was described in the earlier reports.

The stremmatograph records were read with a binocular microscope fitted with a micrometer eyepiece. Readings were taken for points in the record corresponding to a wheel over the instrument and to a point between wheels, the

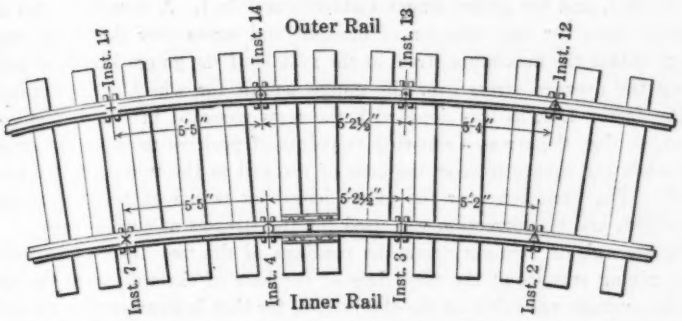


FIG. 5.—POSITION OF EIGHT STREMMATOGRAPHS IN A TEST OF CURVED TRACK.

high point of the record being assumed to have been made when the wheel was directly over the instrument. In reducing these measurements, the readings were multiplied by the proper microscopic constant and then reduced to

TABLE 9.—WEIGHTS OF ROTATING AND RECIPROCATING PARTS, IN POUNDS, MIKADO TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

RECIPROCATING PARTS.

Piston	1 097
Cross-head	570
Weight of main rod on cross-head pin	405
	2 072

$$\frac{2 \times 2\,072}{3 \times 4} = 345$$

ROTATING PARTS.

Driver number	4	Main.	2	1
Two-thirds of weight of reciprocating parts	345	345	345	345
Main rod, weight on crank-pin	....	670	....	....
Side rod, weight on crank-pin	180	392	308	135
Crank-pin	83	406	77	67
Crank-pin hub	135	210	135	127
One-half eccentric crank	....	66	....	....
Total	693	2 089	865	674
Equivalent weight required at center of counterweight	530	2 089	661	515
Weight used	530	2 089	661	515
Difference between weight required and weight obtained	0	0	0	0
Equivalent difference at crank-pin circle	0	0	0	0
Overbalance or underbalance at crank-pin circle for rotating parts only	+345	+345*	+345	+345

*If the effect of the rods and pin not being in the plane of the counterweight is calculated by the method given in the second progress report, this becomes approximately an overbalance of 95 lb.

stresses by multiplying by a constant which involves the position of the neutral axis of the rail section, the vertical distance of the needle-bar below the base of the rail, the modulus of elasticity of steel (taken as 30 000 000 lb. per sq. in.), and the gauge length (which was 4 in.). A correction was also made to allow for the variation of moment and stress over the gauge length and to obtain the maximum stress at the middle of the gauge length by multiplying the average stress over the gauge length for wheel over instrument by the factor 1.04, as was done with the tests recorded in the first progress report, so that the stresses reported at points of positive moment are stresses in pounds per square inch at the base of the rail at the middle of the gauge length. The variation over the gauge length at points of negative moment was slight, and no correction was used for the stresses at such points.

In general, on straight track the readings of the two disks of an instrument, giving stresses at the two edges of the base of the rail, were averaged, and the average was taken as the observation for that instrument for the given run for the given position of wheel. On straight track, in case the record of one disk of an instrument was defective and that on the companion disk was clear, the one good record was usually also discarded. In the case of curved track, all readable records were used, even though the record on the companion disk was defective. It was desired to keep the strains at the two edges separated. It was also found that on curved track there was less vibration in the rail and less opportunity for errors being introduced through the use of a record at only one edge. Besides, it was found that defective records were less frequent on curved track, usually less than 10% of the points having to be discarded.

The accuracy of the records and their reduction is considered to be as good as that of the stremmatograph data reported in the two earlier reports.

The time required for the reduction of the data of the large number of observations made in all the tests was considerable. The reading of the stremmatograph records, the calculation of the data, and the representation of the results for study and interpretation involved a large amount of work, as may be judged from a consideration of the number of microscopic readings of strains made, approximately 470 000. With clear records, skilled observers learned to make 1 000 to 1 200 such readings per day.

Table 1 gives the properties of the sections of the rails new. Profiles were taken of the sections of the rail in the track. With the exception of the 105-lb. rail on the Delaware, Lackawanna and Western Railroad, the rails were not worn enough to have a sufficiently marked effect on the moment of inertia and section modulus about the horizontal axis to make it seem necessary to use the section modulus of the worn rail in the analytical calculation of stresses in rail. On the 7½° curve, the moment of inertia and section modulus of the inner rail were reduced about 3% and of the outer rail about 6%, but, in this case, also, the properties of the new rail have been used in making comparisons. The effect of wear on the section modulus about a vertical axis would be small, as the base of rail, which is not changed, forms the large

factor in making up the section modulus. For lateral bending of the rail on curved track, the properties of new rails were also used.

II.—Tests on Straight Track

6.—Results of Tests.—The tests on straight track, although made for the purpose of supplying a basis of comparison with results on curved track, furnish considerable additional information on the action of straight track, particularly on the effect of speed and counterbalance with several types of locomotives.

To determine the stiffness of track and the value of the modulus of elasticity of rail support, u, measurements of track depression under the truck of loaded cars were made. For the significance of the term, modulus of elasticity of rail support, see the first progress report of the Committee.* The method of making the test was similar to that described in the first progress report, the level bar being used.

On the Atchison, Topeka and Santa Fe Railway, the lighter of the two loads applied was an empty car, and the heavier load was a heavily loaded coal car. On the Delaware, Lackawanna and Western Railroad, the lightest load was an empty car, the medium load a loaded coal car, and the heaviest load a coal car heavily loaded with ore. In each case, the load considered was that on one truck at an end of the car. The load on the wheels of the truck used was carefully weighed, a check on the weight being obtained by also weighing the load on the other truck and the load on the whole car.

Figs. 6 and 7 give the results of tests of track depression on the Atchison, Topeka and Santa Fe Railway, at Ribera, N. Mex., and Figs. 8 and 9, the results of tests on the track of the Delaware, Lackawanna and Western Railroad, at Dover, N. J. Location A on the Atchison, Topeka and Santa Fe Railway covered the location of the stremmatographs on the regular tests at Ribera; Location B was 130 ft. east of Location A. Location C coincided with the location of the stremmatographs at Dover; and Location D was 260 ft. east of Location C. The track-depression profiles indicate that the track on which the tests were made was in fairly good condition, the "play curve" showing uniformity of play between rail and tie and tie and ballast, and uniformity of stiffness in each case.

The value of the modulus of elasticity of rail support, u, calculated from these track-depression profiles by the method described in the first progress report, was 1 600 lb. per lin. in. of rail, for the track on both the Atchison, Topeka and Santa Fe Railway and the Delaware, Lackawanna and Western Railroad. These values indicate a fairly stiff track.

In all the tests to find the stresses in the rail, four stremmatographs were placed on one rail and four on the other, the four instruments on one rail being spaced at distances of about 66 in., and the set on the other rail being placed directly opposite. For all stress diagrams on straight track, the stress given is the mean stress in base of rail, that is, the average of the stresses

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1204; *Proceedings*, Am. Ry. Eng. Assoc., Vol. 19, p. 884.

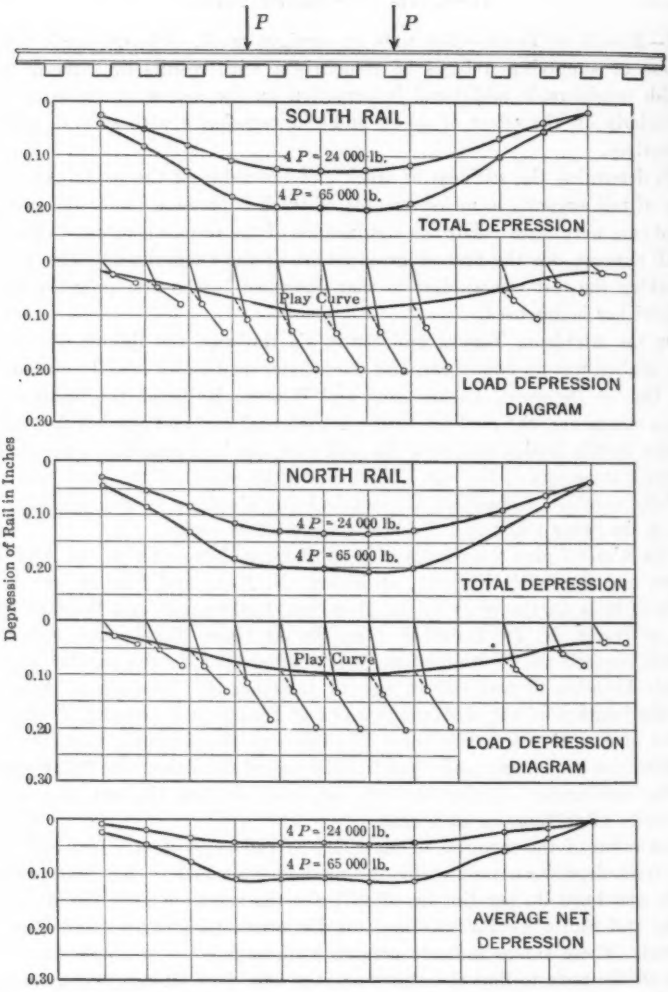


FIG. 6.—TRACK DEPRESSION PROFILES, STATIC LOAD TESTS ON THE ATCHISON, TOPEKA AND SANTA FE RAILWAY, LOCATION *A*.

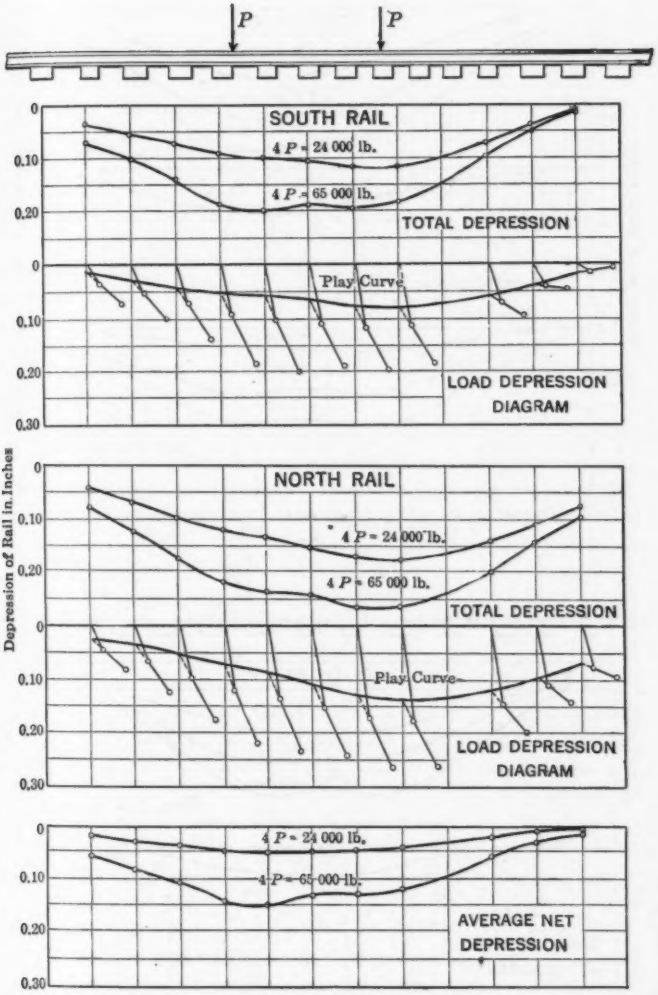


FIG. 7.—TRACK DEPRESSION PROFILES, STATIC LOAD TESTS ON THE ATCHISON, TOPEKA AND SANTA FE RAILWAY, LOCATION *B*.

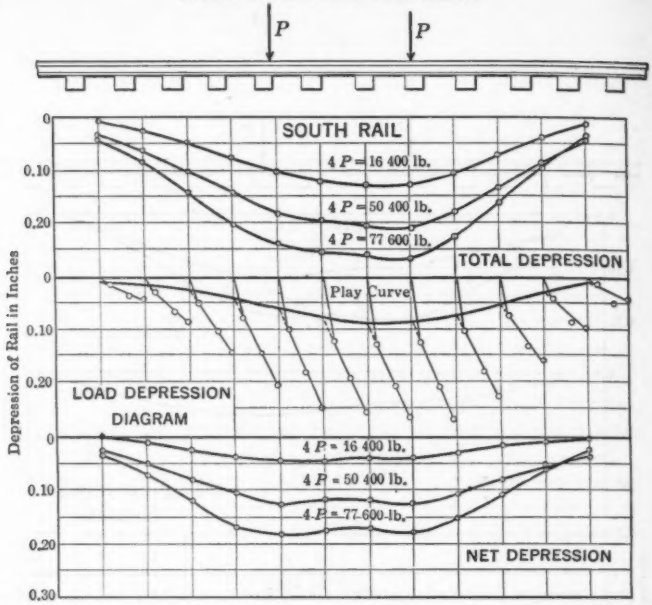


FIG. 8.—TRACK DEPRESSION PROFILES, STATIC LOAD TESTS ON THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD, LOCATION *C*.

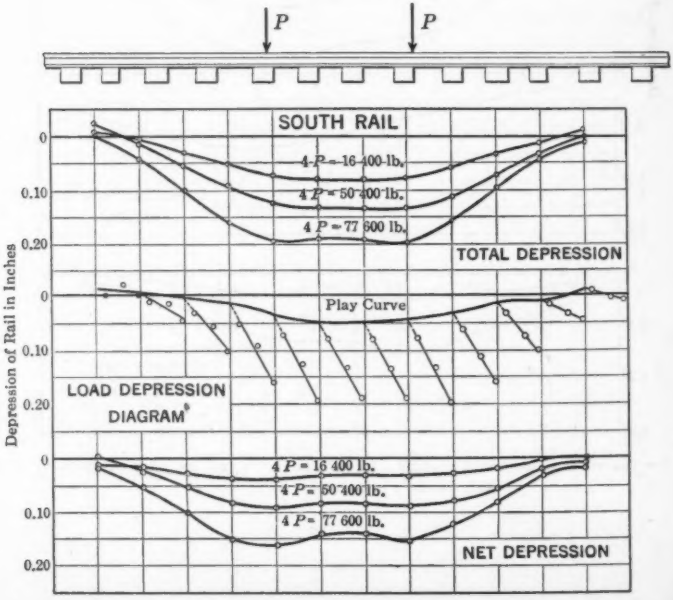


FIG. 9.—TRACK DEPRESSION PROFILES, STATIC LOAD TESTS ON THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD, LOCATION *D*.

at the outside and inside edges of the rail, and the stress there recorded is the average of the results on both rails as found from the records of the eight instruments. In Figs. 10, 11, and 12 are given the mean stress in the base of rail for the high and the low position of the counterweight for the Pacific type locomotive, the Light Prairie type, and the Balanced Compound Prairie type, respectively, in the tests of track of the Atchison, Topeka and Santa Fe Railway at East Fort Madison, Ill. In Figs. 13, 14, 15, 16, 17, and 20 are given the corresponding stresses for the tests with the Mountain type locomotive coasting down grade, the Mountain type running up grade, the Light Santa Fe type and the Heavy Santa Fe type coasting down grade, the Heavy Santa Fe type working steam down grade and the Double Trailer Heavy Santa Fe type coasting down grade, respectively, at Ribera, N. Mex.

In Tables 10 to 18, inclusive, are given the average or mean values of the stresses in base of rail under the locomotive wheels found in the tests, and also the values at the high point and the low point of the counterweight. The calculated stresses under the nominal static wheel loads are also given. The calculations are based on the method of analysis given in the first progress report, and the value of u from the tests of track depression was used. It will be noted that the calculated values of the stresses for static loads in general do not differ greatly from the observed values at a speed of 5 miles per hour. Where there is a noticeable difference, it seems probable that the actual distribution of load among the individual wheels is not the same as the assumed distribution. It is found, however, that the average of the calculated stress under the several wheels of the locomotive is almost exactly the same as the average of the observed stresses under the wheels at a speed of 5 miles per hour, the greatest difference in all the tests being about 6 per cent.

TABLE 10.—STRESSES IN RAIL WITH PACIFIC TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY. MAXIMUM, MINIMUM, AND MEAN VALUES FROM CURVES FOR SERIES 5450-5458.

Stresses are given in pounds per square inch at base of rail.

Speed, in miles per hour.	Position of counterweight.	Trailer.	Driver Number.			Front Truck Wheels.	
			3	Main.	1	2	1
5	Mean value	20 700	18 900	17 700	17 500	6 000	10 400
25	Up	.....	18 000	.....	.....	.....	.....
	Down	.....	23 000	.....	.....	.....	.....
	Mean value	21 400	20 200	18 800	19 200	6 900	11 800
40	Up	.....	16 500	22 000	18 500	.....	.....
	Down	.....	26 500	15 500	23 000	.....	.....
	Mean value	21 800	21 000	19 100	20 500	7 700	12 800
60	Up	.....	17 500	27 000	17 500	.....	.....
	Down	.....	29 000	18 500	26 500	.....	.....
	Mean value	23 400	23 500	23 100	22 200	9 800	13 800
Calculated stress under static load		22 900	19 600	15 000	17 000	4 600	9 400
Calculated additional stress due to counterbalance at 60 miles per hour		.....	+5 900	+4 700	+5 900	.....	.....

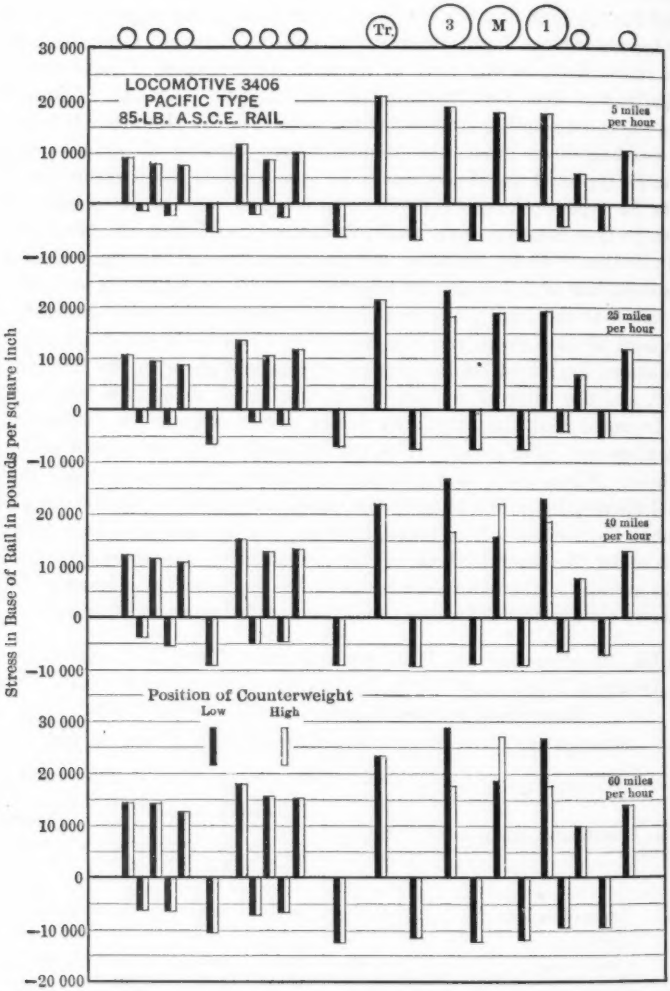


FIG. 10.—STRESS IN RAIL ON STRAIGHT TRACK AT HIGH AND LOW POSITION OF COUNTERWEIGHT, SERIES 5450–5458, PACIFIC TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

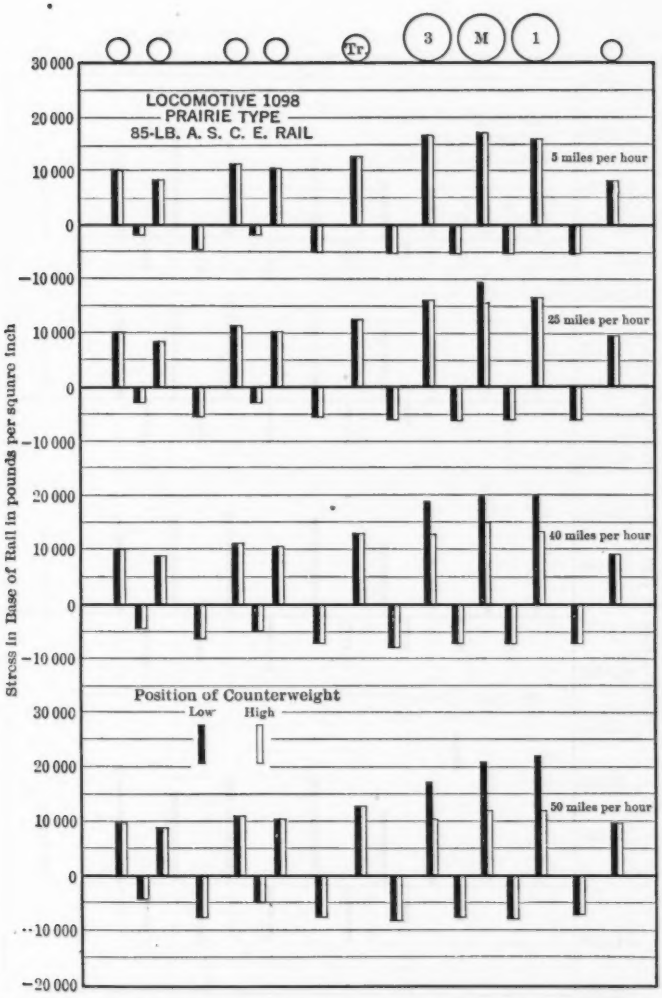


FIG. 11.—STRESS IN RAIL ON STRAIGHT TRACK AT HIGH AND LOW POSITION OF COUNTERWEIGHT, SERIES 5441–5449, LIGHT PRAIRIE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

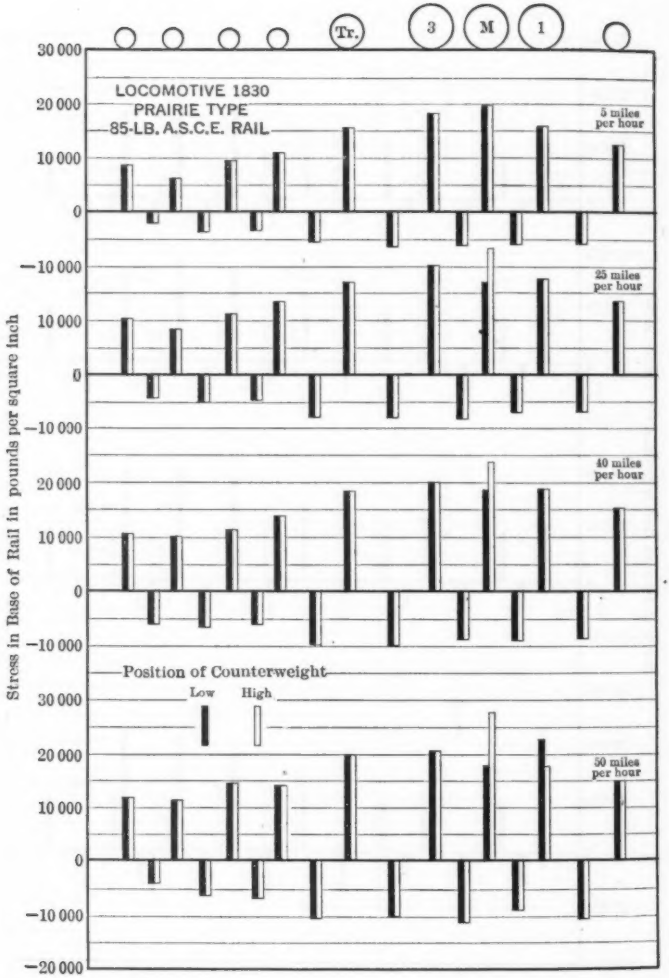


FIG. 12.—STRESS IN RAIL ON STRAIGHT TRACK AT HIGH AND LOW POSITION OF COUNTERWEIGHT, SERIES 5434–5440, BALANCED COMPOUND PRAIRIE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

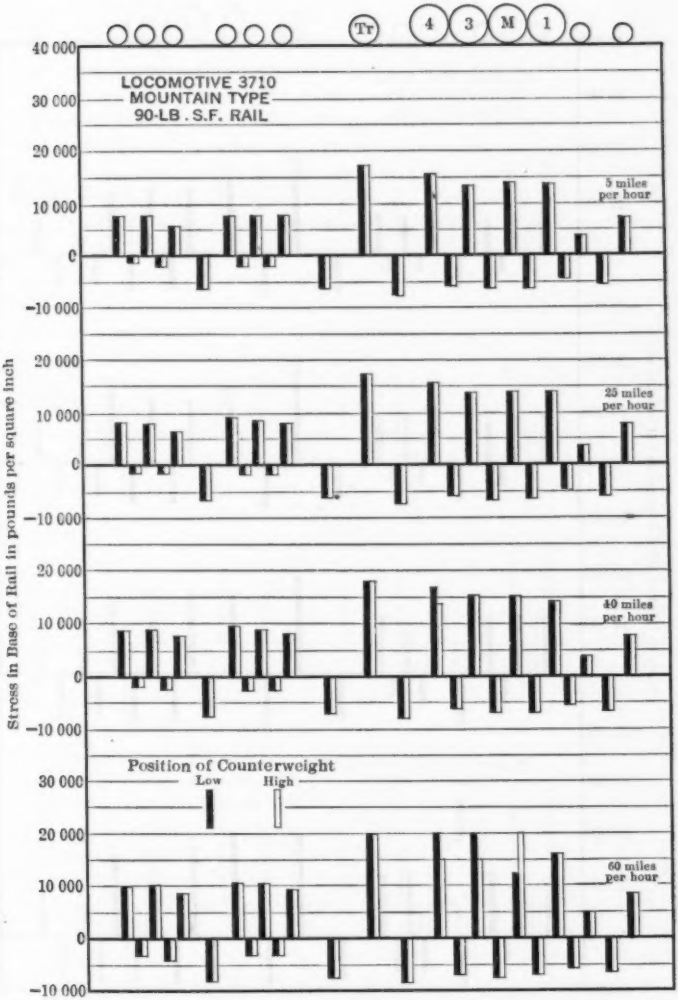


FIG. 13.—STRESS IN RAIL ON STRAIGHT TRACK AT HIGH AND LOW POSITION OF COUNTERWEIGHT, SERIES 5338–5344, MOUNTAIN TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

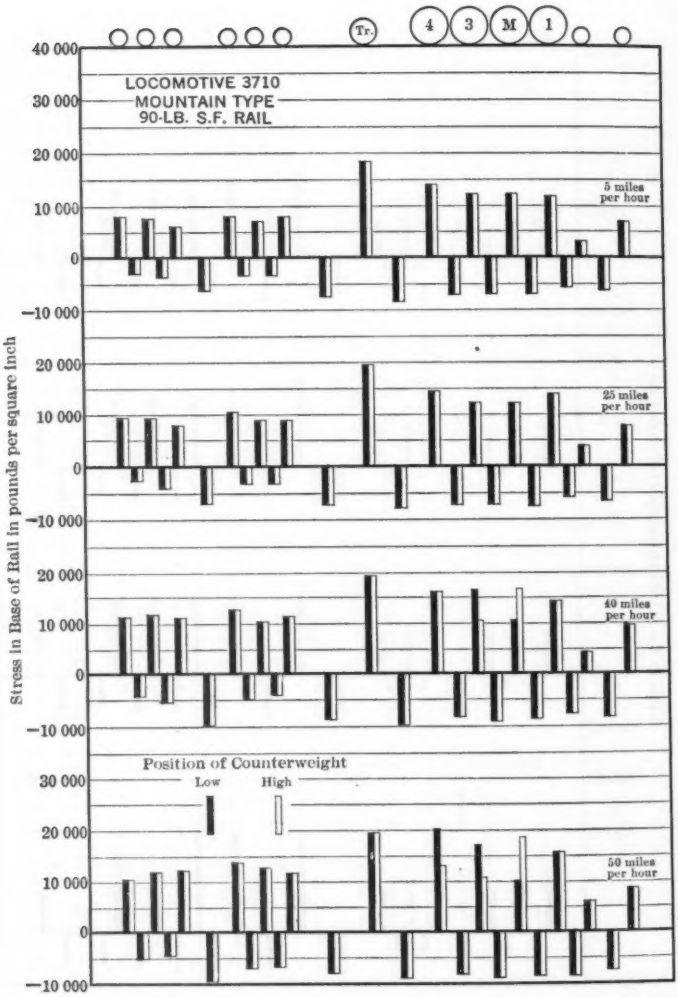


FIG. 14.—STRESS IN RAIL ON STRAIGHT TRACK AT HIGH AND LOW POSITION OF COUNTERWEIGHT, SERIES 5345–5350, MOUNTAIN TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

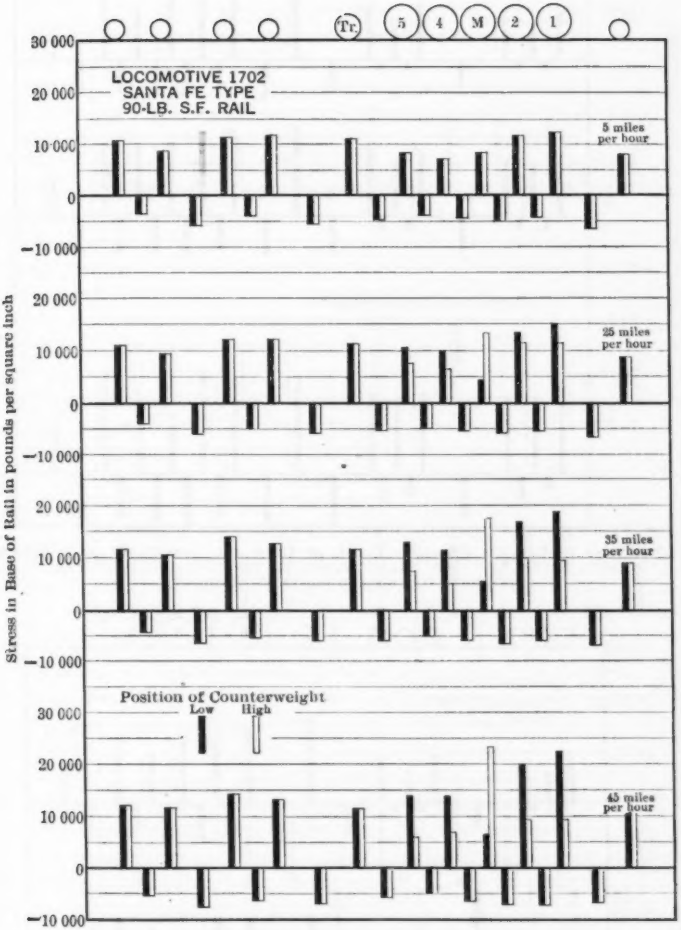


FIG. 15.—STRESS IN RAIL ON STRAIGHT TRACK AT HIGH AND LOW POSITION OF COUNTERWEIGHT, SERIES 5300–5318, LIGHT SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

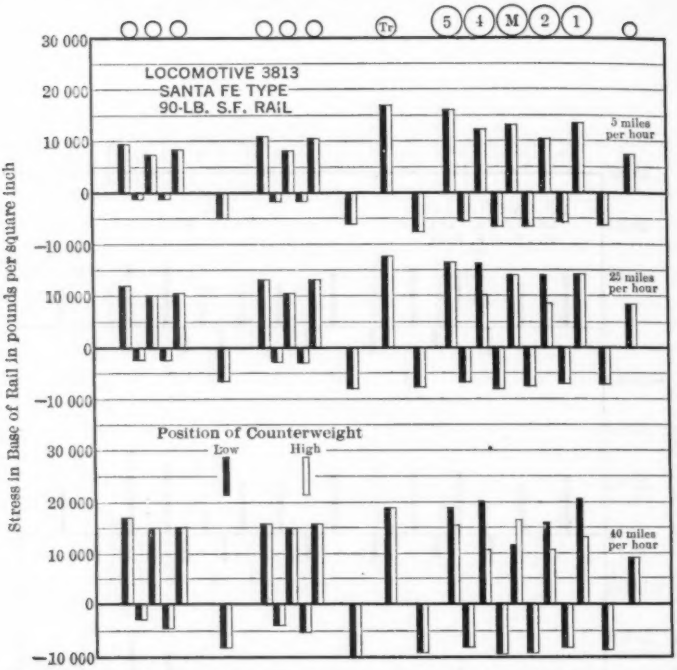


FIG. 16.—STRESS IN RAIL ON STRAIGHT TRACK AT HIGH AND LOW POSITION OF COUNTERWEIGHT, SERIES 5401–5408, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

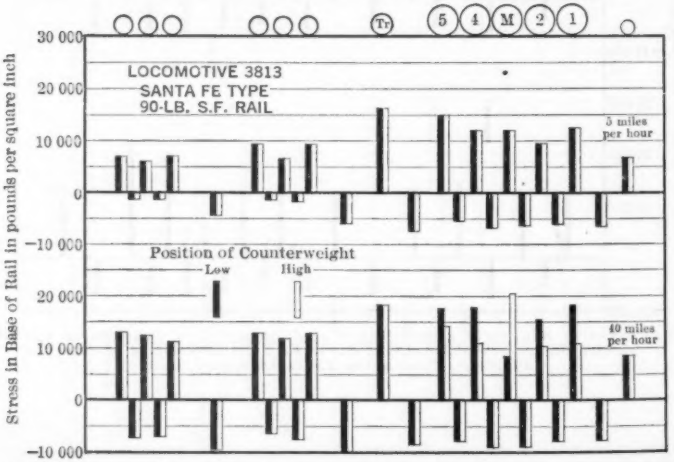


FIG. 17.—STRESS IN RAIL ON STRAIGHT TRACK AT HIGH AND LOW POSITION OF COUNTERWEIGHT, SERIES 5409–5415, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

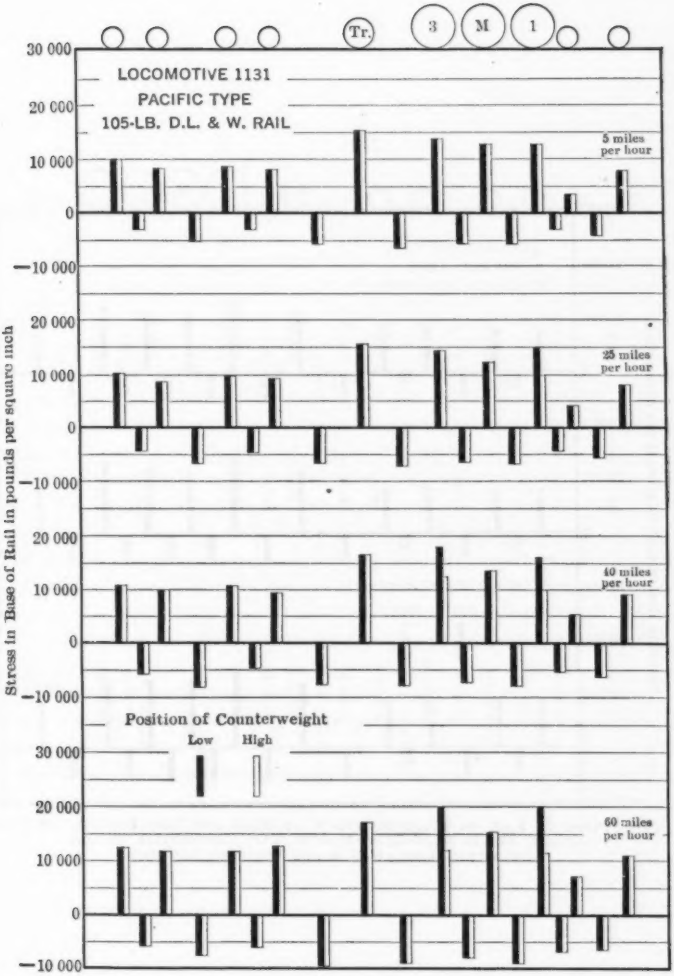


FIG. 18.—STRESS IN RAIL ON STRAIGHT TRACK AT HIGH AND LOW POSITION OF COUNTERWEIGHT, SERIES 5136–5149, PACIFIC TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD

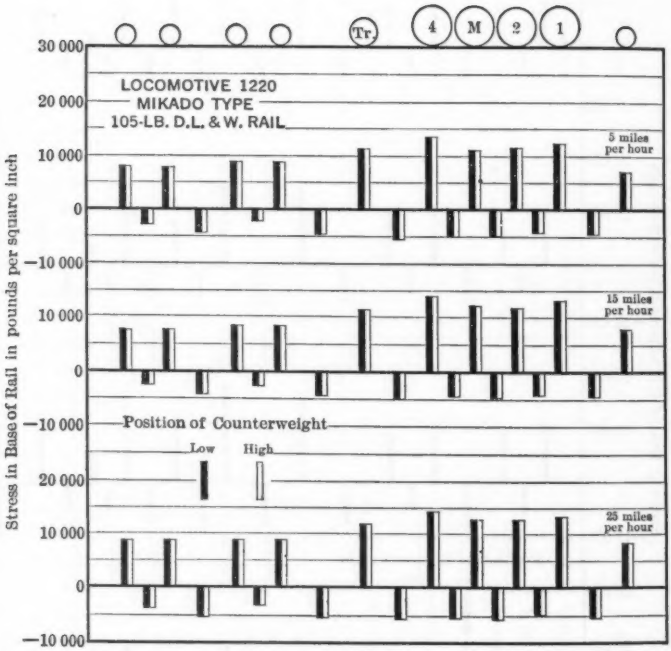


FIG. 19.—STRESS IN RAIL ON STRAIGHT TRACK AT HIGH AND LOW POSITION OF COUNTERWEIGHT, SERIES 5206–5220, MIKADO TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

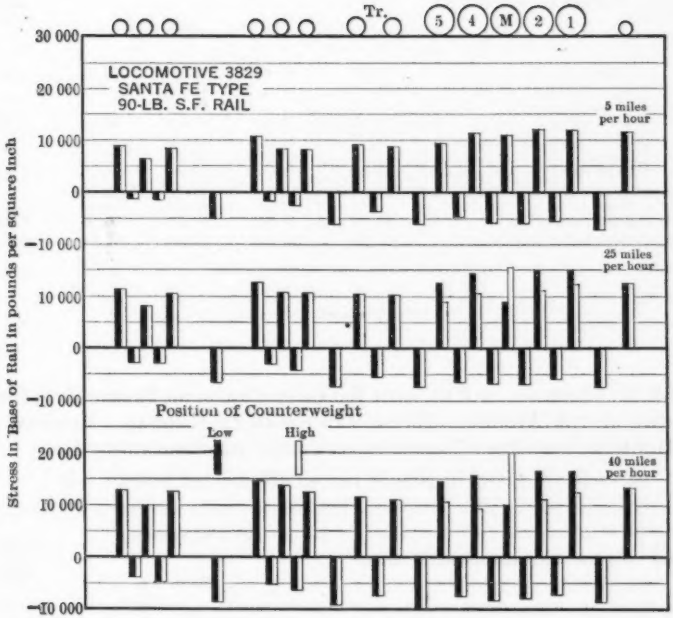


FIG. 20.—STRESS IN RAIL ON STRAIGHT TRACK AT HIGH AND LOW POSITION OF COUNTERWEIGHT, SERIES 5416–5424, DOUBLE TRAILER HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

TABLE 11.—STRESSES IN RAIL WITH LIGHT PRAIRIE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY. MAXIMUM, MINIMUM, AND MEAN VALUES FROM CURVES FOR SERIES 5441-5449.

Stresses are given in pounds per square inch at base of rail.

Speed, in miles per hour.	Position of counterweight.	Trailer.	Driver Number.			Front truck wheel.
			3	Main.	1	
5	Mean value	12 600	16 400	16 900	15 800	8 200
25	Up	.....	.....	15 500	.....	.....
	Down	.....	.....	19 500	.....	.....
	Mean value	12 400	16 100	17 300	16 400	9 600
40	Up	.....	13 000	15 000	13 500	.....
	Down	.....	19 000	20 000	20 000	.....
	Mean value	13 100	16 100	16 700	15 700	9 300
50	Up	.....	10 500	12 000	12 000	.....
	Down	.....	17 500	21 000	22 000	.....
	Mean value	12 700	14 200	16 400	16 800	9 700
Calculated stress under static load		14 200	13 700	14 000	14 300	10 000

TABLE 12.—STRESSES IN RAIL WITH BALANCED COMPOUND PRAIRIE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY. MAXIMUM, MINIMUM, AND MEAN VALUES FROM CURVES FOR SERIES 5434-5440.

Stresses are given in pounds per square inch at base of rail.

Speed, in miles per hour.	Position of counterweight.	Trailer.	Driver Number.			Front truck wheel.
			3	Main.	1	
5	Mean value	15 300	18 400	20 000	15 800	12 500
25	Up	.....	.....	23 000*	.....	.....
	Down	.....	.....	17 000*	.....	.....
	Mean value	16 800	19 800	20 200	17 400	13 500
40	Up	.....	.....	23 500*	.....	.....
	Down	.....	.....	18 500*	.....	.....
	Mean value	18 200	19 800	20 000	18 700	15 100
50	Up	.....	.....	27 500*	17 500	.....
	Down	.....	.....	18 000*	22 500	.....
	Mean value	19 700	20 200	21 300	20 000	14 800
Calculated stress under static load		14 700	18 800	15 100	18 700	11 200

* The maximum and minimum occur at the quarter-points.

TABLE 13.—STRESSES IN RAIL WITH MOUNTAIN TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY. MAXIMUM, MINIMUM, AND MEAN VALUES FROM CURVES FOR SERIES 5338-5344.

Stresses are given in pounds per square inch at base of rail

Speed, in miles per hour.	Position of counterweight.	Trailer	Driver Number.				Front Truck Wheels.	
			4	3	Main.	1	2	1
5	Mean value	17 000	15 300	13 300	14 000	13 600	4 000	7 500
25	Up	.....	.....	.....	.....	.....	.....	.....
	Down	.....	.....	.....	.....	.....	.....	.....
	Mean value	17 400	15 400	13 900	14 100	14 200	3 800	8 000
40	Up	.....	14 000	.....	.....	.....	.....	.....
	Down	.....	17 000	.....	.....	.....	.....	.....
	Mean value	18 200	15 600	15 400	15 200	14 400	4 000	7 900
60	Up	.....	15 000	15 000	20 000	.....	.....	.....
	Down	.....	20 000	20 000	12 500	.....*	.....	.....
	Mean value	19 500	17 200	17 300	17 100	15 900	5 000	8 500
Calculated stress under static load		21 200	14 400	12 100	11 400	14 000	2 800	7 300
Calculated additional stress due to counterbalance at 60 miles per hour		.....	+4 700	+3 500	+3 500	+4 700	.....	.....

*Not sufficient tests to determine the stress due to counterbalance.

TABLE 14.—STRESSES IN RAIL WITH LIGHT SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY. MAXIMUM, MINIMUM, AND MEAN VALUES FROM CURVES FOR SERIES 5300-5318.

Stresses are given in pounds per square inch at base of rail.

Speed, in miles per hour.	Position of counterweight.	Trailer.	Driver Number.					Front truck wheel.
			5	4	Main.	2	1	
5	Mean value	10,700	8 100	7 100	8 100	11 500	12 100	7 800
25	Up	.....	7 500	6 500	13 500	11 500	11 500	.....
	Down	.....	10 500	10 000	4 500	13 500	15 000	.....
	Mean value	11 300	9 500	7 900	9 900	12 500	13 100	8 800
35	Up	.....	7 500	5 000	17 500	10 000	19 000	.....
	Down	.....	13 000	11 500	5 500	17 000	9 500	.....
	Mean value	11 600	10 500	8 500	11 500	13 600	14 600	9 200
45	Up	.....	6 000	7 000	23 500	9 500	9 500	.....
	Down	.....	14 000	14 000	6 500	20 000	22 500	.....
	Mean value	11 700	10 200	10 500	14 300	14 600	14 800	10 100
Calculated stress under static load		4 900	10 800	7 900	10 000	8 100	12 200	7 600

TABLE 15.—STRESSES IN RAIL WITH HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY. MAXIMUM, MINIMUM, AND MEAN VALUES FROM CURVES FOR SERIES 5401-5408.

Stresses are given in pounds per square inch at base of rail.

Speed, in miles per hour.	Position of counterweight.	Trailer.	Driver Number.					Front truck wheel.
			5	4	Main.	2	1	
5	Mean value	16 900	15 900	12 200	12 900	10 100	13 400	7 300
25	Up	.....	.....	10 000	.....	8 500	.....	.....
	Down	.....	.....	16 000	.....	14 000	.....	.....
	Mean value	17 700	16 400	13 600	13 800	11 100	14 200	8 300
40	Up	.....	15 500	10 500	16 500	10 500	13 000	.....
	Down	.....	19 000	20 000	11 500	16 000	20 500	.....
	Mean value	18 900	17 200	14 700	14 200	12 700	16 300	8 800
Calculated stress under static load		19 700	15 400	11 300	11 000	10 600	14 400	6 700
Calculated additional stress due to counterbalance at 40 miles per hour		.....	+3 700	+3 600	−2 800	+3 600	+3 700	.....

TABLE 16.—STRESSES IN RAIL WITH DOUBLE TRAILER HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY. MAXIMUM, MINIMUM, AND MEAN VALUES FROM CURVES FOR SERIES 5416-5424.

Stresses are given in pounds per square inch at base of rail.

Speed, in miles per hour.	Position of counterweight.	Trailer.		Driver Number.					Front truck wheel.
		2	1	5	4	Main.	2	1	
5	Mean value	9 200	8 900	9 500	11 400	11 100	11 900	11 900	11 700
25	Up	.....	.....	9 000	10 500	15 500	11 000	12 500	.....
	Down	.....	.....	12 500	14 500	9 000	15 000	15 000	.....
	Mean value	10 400	10 100	10 100	12 400	12 200	13 000	13 900	12 400
40	Up	.....	.....	10 500	9 500	20 000	11 000	12 500	.....
	Down	.....	.....	14 500	15 500	10 000	16 500	16 500	.....
	Mean value	11 500	11 100	12 400	12 800	12 900	14 000	14 900	13 300
Calculated stress under static load.		8 700	7 500	12 600	9 500	10 000	11 300	14 500	8 600
Calculated additional stress due to counterbalance at 40 miles per hour		.....	.....	+3 700	+3 600	−2 800	+3 600	+3 700	.....

TABLE 17.—STRESSES IN RAIL WITH PACIFIC TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD. MAXIMUM, MINIMUM, AND MEAN VALUES FROM CURVES FOR SERIES 5136-5149.

Stresses are given in pounds per square inch at base of rail.

Speed, in miles per hour.	Position of counterweight.	Trailer.	Driver Number.			Front Truck Wheels.	
			3	Main.	1	2	1
5	Mean value.....	15 300	13 700	12 600	12 600	3 300	7 700
25	Up.....	.....	.....	.....	10 000	.....	.....
	Down.....	.....	.....	.....	15 000	.....	.....
	Mean value.....	15 700	14 300	12 400	12 300	4 200	7 900
40	Up.....	.....	12 500	.....	10 000	.....	.....
	Down.....	.....	18 000	.....	16 000	.....	.....
	Mean value.....	16 600	15 000	13 600	13 000	5 200	9 000
60	Up.....	.....	12 000	.....	11 500	.....	.....
	Down.....	.....	20 000	.....	20 000	.....	.....
	Mean value.....	17 400	15 800	15 300	15 700	7 100	10 900
Calculated stress, under static load.....		15 800	14 800	12 200	14 600	1 500	5 400
Calculated additional stress due to counterbalance at 60 miles per hour.....		.....	+5 900	+4 500	+5 900	.....	.....

TABLE 18.—STRESSES IN RAIL WITH MIKADO TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD. MAXIMUM, MINIMUM, AND MEAN VALUES FROM CURVES FOR SERIES 5206-5220.

Stresses are given in pounds per square inch at base of rail.

Speed, in miles per hour.	Position of counterweight.	Trailer.	Driver Number.				Front truck wheel.
			4	Main.	2	1	
5	Mean value.....	11 400	13 600	11 100	11 500	12 200	7 000
15	Up.....	.....	.....	.....	.....	.....	.....
	Down.....	.....	.....	.....	.....	.....	.....
	Mean value.....	11 400	14 100	12 000	11 900	12 000	7 800
25	Up.....	.....	.....	.....	.....	.....	.....
	Down.....	.....	.....	.....	.....	.....	.....
	Mean value.....	11 900	14 200	12 500	12 700	13 100	8 200
Calculated stress under static load.....		12 600	12 200	10 100	10 100	13 000	6 500

Attention should be called to the fact that the stresses given in the diagrams and tables are average stresses, the result of averaging a number of observations, usually a large number, from 50 to 125. This means that many observations lie above the average value and many below it. In fact, when all the observations are plotted for a revolution of a driver, there is a belt of points within two rather definite boundaries, one on each side of the line of averages, as illustrated in Figs. 21 to 25, inclusive. Within these boundaries, the points are fairly well distributed, and outside of them a few scattered points will be found. For straight track, the boundaries of the belt of points may be 5 000 lb. per sq. in. above and below the average line.

From an inspection of the diagrams and tables, it will be noted that at a speed of 5 miles per hour the average stress in the base of rail of the 85-lb. Am. Soc. C. E. section, ranged from 15 800 to 20 000 lb. per sq. in. under the drivers of the Pacific type locomotive and the two Prairie types of the Atchison, Topeka and Santa Fe Railway. On the 90-lb. S. F. rail, the average stress ranged from 10 100 to 15 900 lb. per sq. in. under the drivers of the Mountain type locomotive and Heavy Santa Fe types, at 5 miles per hour. For the Light Santa Fe type, having much lighter wheel loads, the corresponding stresses in the 90-lb. S. F. rail, at the same speed, range from 7 100 to 12 100 lb. per sq. in. For the Pacific and Mikado types of the Delaware, Lackawanna and Western Railroad, the stresses in the 105-lb. D. L. & W. section under the drivers, at 5 miles per hour, ranged from 11 100 to 13 700 lb. per sq. in. It should be borne in mind that the stress named in every case is the average value found in a considerable number of runs and that when a locomotive is run over the track many times, individual stresses will be found under a given wheel that are considerably greater in magnitude than the average stress named. It should also be remembered that the lateral bending of the rail under the action of the locomotive on straight track will result in stresses at the outside or the inside edge of the base of rail that are materially higher than the mean of the stresses at the two edges, and, also, that the effect of speed and counterbalance is to add considerably to the values found at 5 miles per hour.

7.—*Effect of the Locomotive Counterbalance.*—In the second progress report of the Committee the problem of counterbalancing the locomotive is briefly considered.* For clearness of expression, the term, counterweight, is there and here used as the weight applied or added to the driver at a point opposite the crank-pin for the purpose of balancing or helping to balance the rotating and reciprocating parts, and the term, counterbalance, as the general condition of underbalance or overbalance as it exists in a given driver, while counterbalancing is used to refer to the general problem of endeavoring to balance the rotating and reciprocating parts.

The vertical effect of a lack of balance of the rotating parts is to increase or decrease the pressure of the drivers on the rails and to vary the pressure upward on the equalizing levers and the frame of the locomotive. Since the counterweight cannot be made to neutralize both the vertical and the horizontal

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 1415; *Proceedings*, Am. Ry. Eng. Assoc., Vol. 21, p. 652.

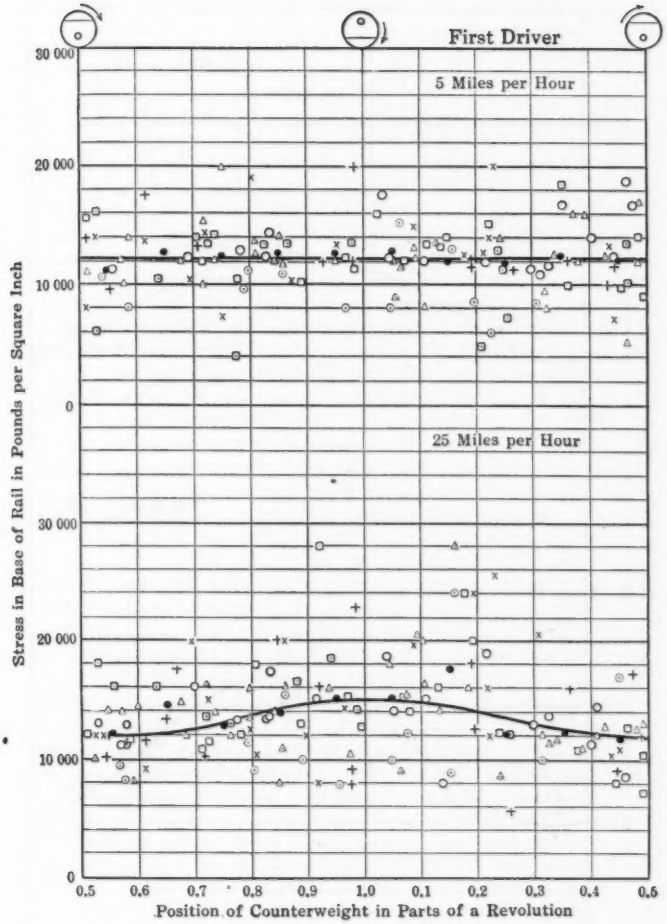


FIG. 21.—OBSERVED VALUES OF STRESS IN RAIL ON STRAIGHT TRACK, SERIES 5300–5318, LIGHT SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

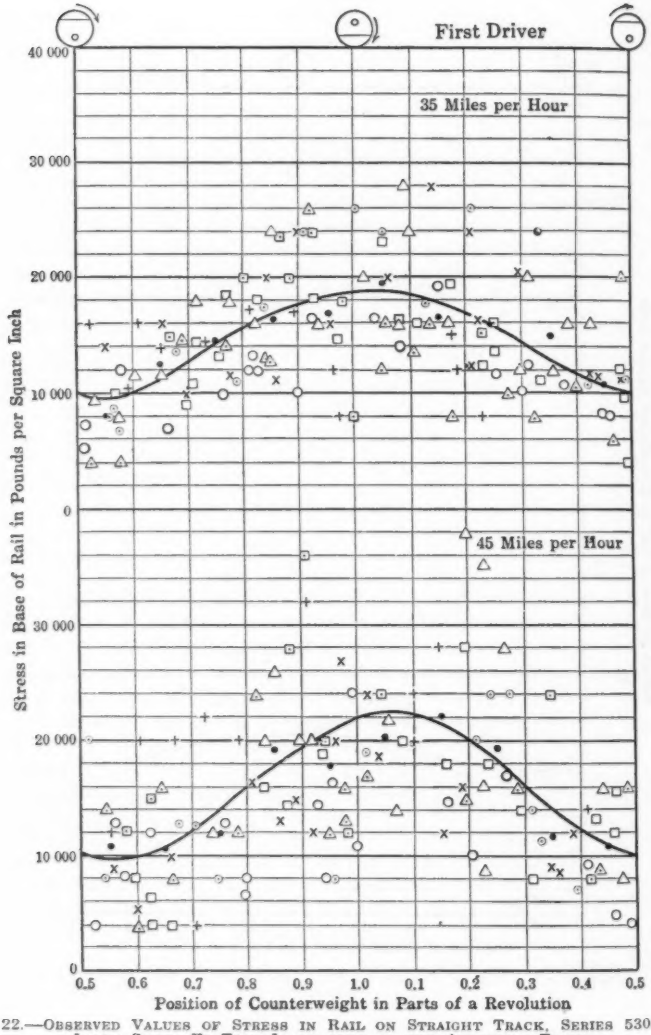


FIG. 22.—OBSERVED VALUES OF STRESS IN RAIL ON STRAIGHT TRACK, SERIES 5300–5318, LIGHT SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

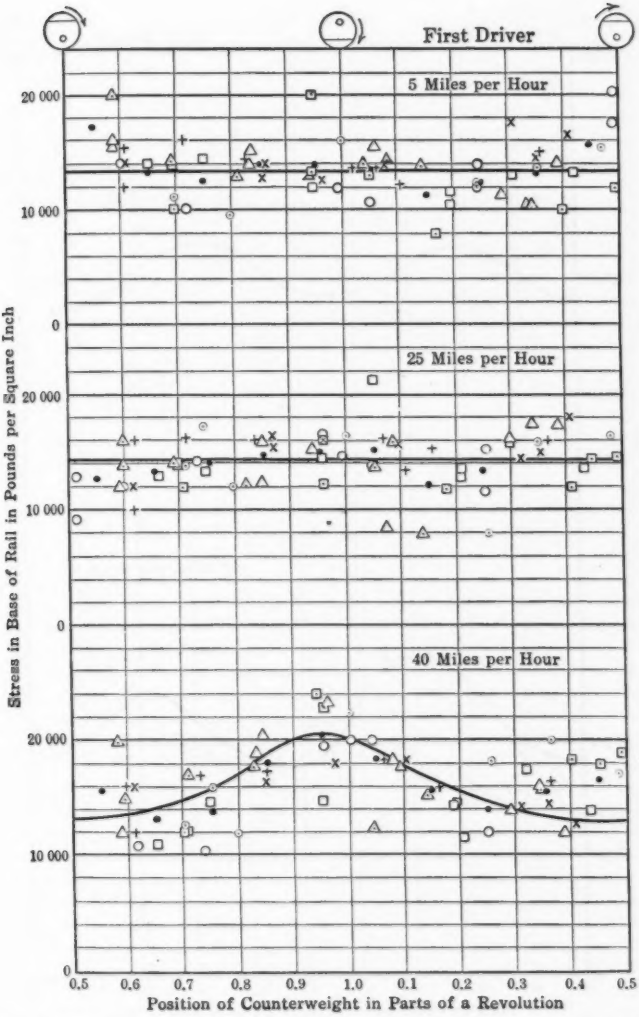


FIG. 23.—OBSERVED VALUES OF STRESS IN RAIL ON STRAIGHT TRACK, SERIES 5401–5408, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

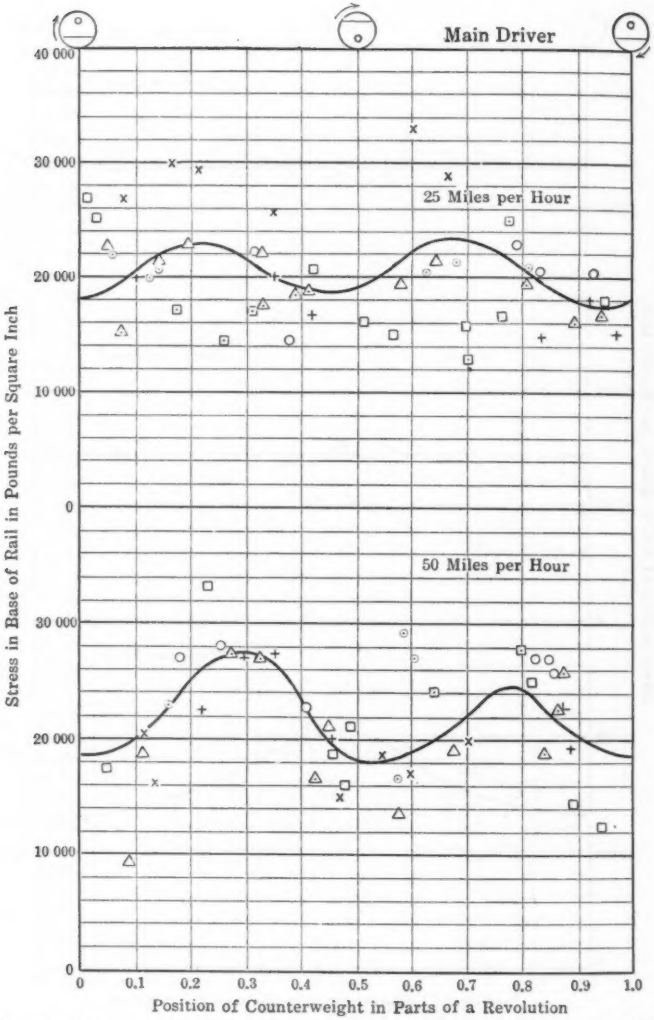


FIG. 24.—OBSERVED VALUES OF STRESS IN RAIL ON STRAIGHT TRACK, SERIES 5434–5440, BALANCED COMPOUND PRAIRIE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

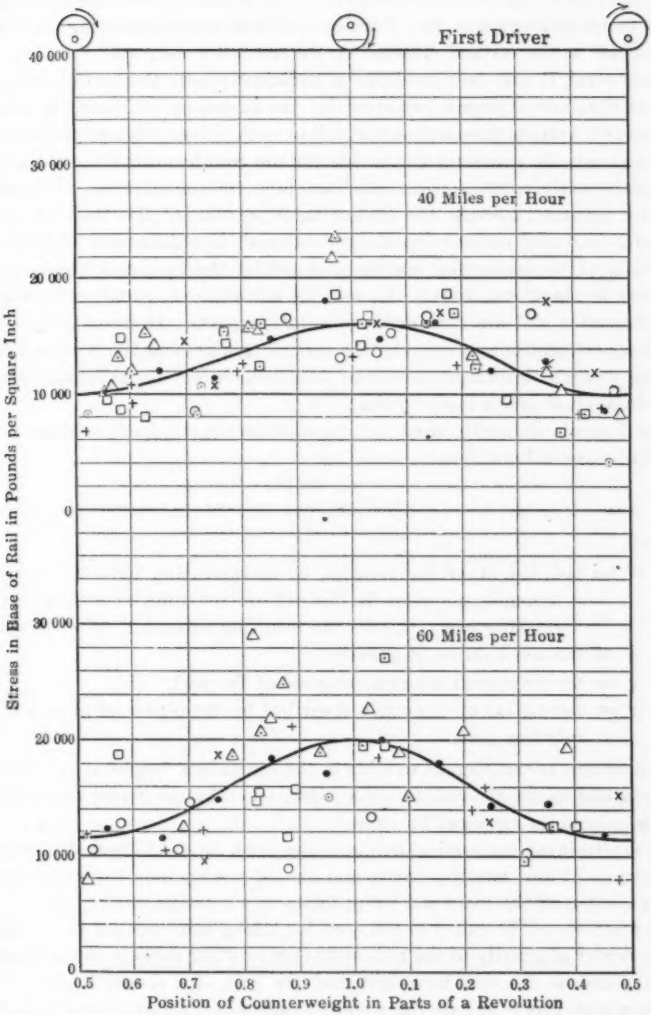


FIG. 25.—OBSERVED VALUES OF STRESS IN RAIL ON STRAIGHT TRACK, SERIES 5136–5149, PACIFIC TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

effect of the moving parts (part of them having horizontal motion only), a compromise must be made in the counterbalancing. It is the common practice in locomotive design to counterbalance for all rotating parts and for a portion of the reciprocating parts, this practice resulting in overbalancing the rotating parts as far as the vertical pressure on the track is concerned.

In general, it may be considered as sufficiently near the truth to count the vertical dynamic augment produced by the excess or deficiency of counterbalance as a vertical pressure on the rail at each driver, with whatever further allowance may be necessary to provide for the counterweight not being in the same plane as the rotating parts which are to be counterbalanced. This method is fairly accurate, because the change in depression of the track due to the added or decreased pressure on the rail is small in comparison with the total depression of the locomotive spring produced by the weight of the locomotive tributary to the given driver. In making calculations, parts of the side rod and connecting rod are considered as rotating parts. It seems proper, then, in making calculations, to consider the vertical dynamic augment of each driver as being separate downward or upward loads which may be added algebraically to the individual driver loads on the rail.

The formula generally used for calculating the dynamic augment is the formula for centrifugal force:

$$C = \frac{W v^2}{g r} \quad \text{.....(12)*}$$

in which,

W = the weight of the rotating or reciprocating part for which the dynamic augment is desired, or the unbalanced part of the rotating parts, in case the resulting rotational effect is wanted.

g = the acceleration of gravity.

v = the rotational velocity, or speed of the part.

r = the radius of the circle described by the center of gravity of the rotating part.

It is convenient to transfer all weights to the equivalent weight at the center of crank-pin and to use the radius of the circle taken by the center of the crank-pin relatively to the center of the driver.

For determining the vertical dynamic augment, W should include the excess or deficiency of the rotating parts, and should not include the reciprocating parts, a portion of the main rod being taken as a rotating part.

Attention should be called to the need for taking into account the condition that the center of gravity of the rods and other rotating parts is not in the same vertical plane as the counterweight and the rail. As shown in the second progress report, this condition has a considerable effect on the counterbalancing of the main drivers, but it is not generally important with the other drivers. A main driver with a seeming overbalance for the rotating parts, as judged by the method of calculation generally used in design, may really be an underbalanced wheel, while for an underbalanced main driver, the deficiency in

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1417.

counterweight may be considerably greater than that expected when this eccentricity of position is not taken into account.

Sample cases of observed values of the stresses in rail throughout the revolution of a driver are plotted in Figs. 21, 22, 23, 24, and 25. The ordinates of the diagrams represent stresses at the base of the rail, in pounds per square inch. The abscissas represent the position of the counterweight at the time the wheel passes over the instrument with respect to its down position, the scale being in fractions of a complete revolution of the wheel. Results given by the several instruments are indicated by individual symbols. The curves representing the stress for a given speed throughout an entire revolution were formed by first averaging the values of the plotted points in groups for each one-tenth of a revolution and then drawing a curve to represent as well as possible the position and trend of these points. As may be expected, at a speed of 5 miles per hour, the variation in stress due to other causes is so much greater than that due to the effect of counterbalance as to mask any effect of counterbalance, and, accordingly, a straight-line average of all the points has been used for all the wheels for a speed of 5 miles per hour. The average of all the observed values, which is approximately the average ordinate for the curves, is called the mean value at the given speed. The difference between this mean value and the high point of the average curve may be taken to be the effect of the counterbalance in increasing the stress (which may be due to overbalance or to underbalance), and the difference in the value to the low point of the curve represents the effect with the counterweight in the opposite position. The position of the counterweight for the maximum and minimum values is also represented on the diagrams.

The same method of determining the average curve of the stress throughout the revolution of a driver was used for all the tests. Figs. 26 to 32, inclusive, give the curves thus found.

As the observations were well distributed throughout the revolution of a driver, the average of the observed values under any wheel at any speed may be taken as the average stress in the rail at that speed without reference to counterbalance effect. The difference between this average stress and the high point and low point of the curve is taken to be the effect of counterbalance.

In Tables 10 to 18, inclusive, are given the stresses with the counterbalance up and counterbalance down and the mean value, and also the calculated additional stress due to counterbalance at one speed, based on the assumed values of counterbalance given in the previous tables, without correction for the outside parts rotating in planes away from the driver.

From the diagrams and tables it will be noted that the highest stress in rail under the main driver in all the locomotives except the Pacific type of the Delaware, Lackawanna and Western Railroad and the Balanced Compound Prairie type of the Atchison, Topeka and Santa Fe Railway occurs when the counterweight is up. For several of the locomotives, a calculation for the counterbalance in the main driver shown in the tables would indicate that, in many cases, the high stress may be expected to occur when the counterweight is down, if the method of calculation is that so generally used of neglecting the

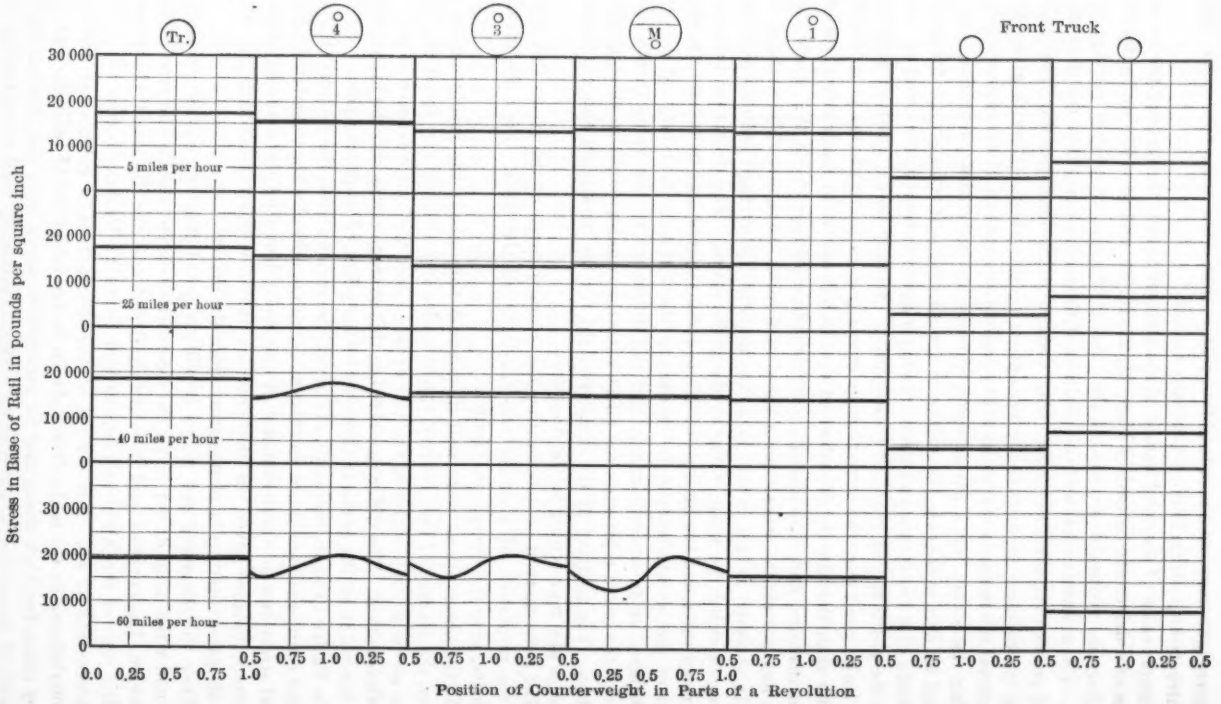


FIG. 26.—CURVES OF AVERAGE STRESS IN RAIL ON STRAIGHT TRACK THROUGHOUT REVOLUTION OF DRIVER, SERIES 5338–5344, MOUNTAIN TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

Locomotive of the Atchison, Topeka and Santa Fe Railway.

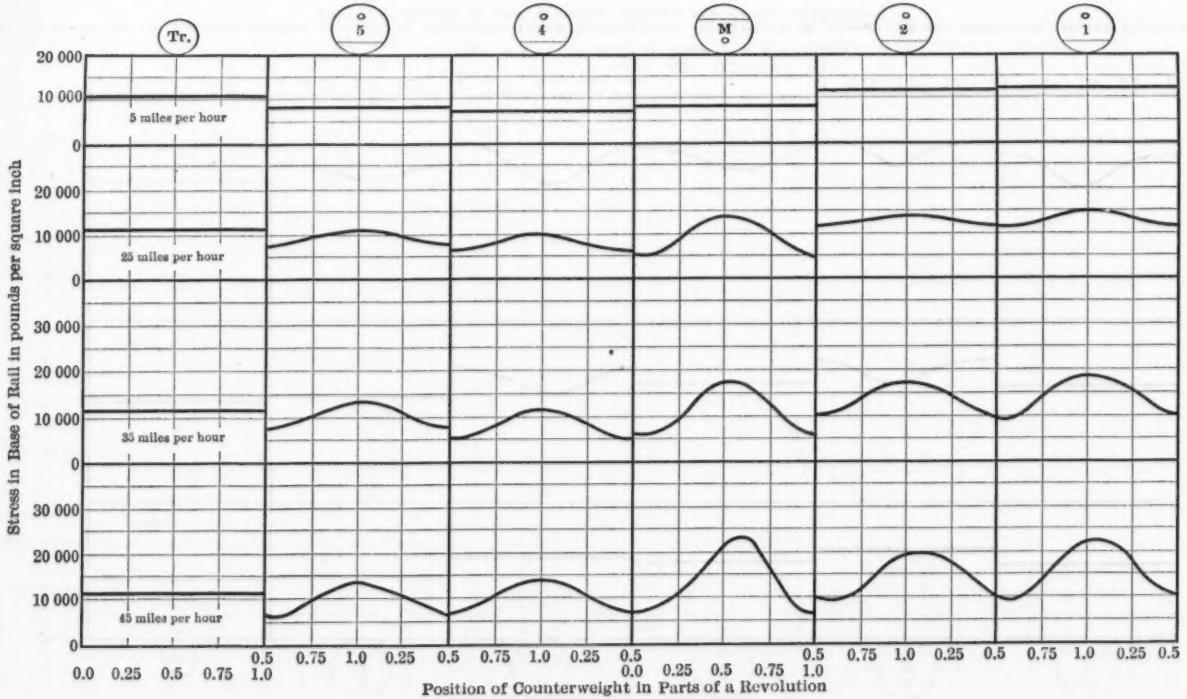


Fig. 27.—Curves of Average Stress in Rail on Straight Track Throughout Revolution of Driver, Series 5300–5318, Light Santa Fe Type Locomotive of the Atchison, Topeka and Santa Fe Railway.

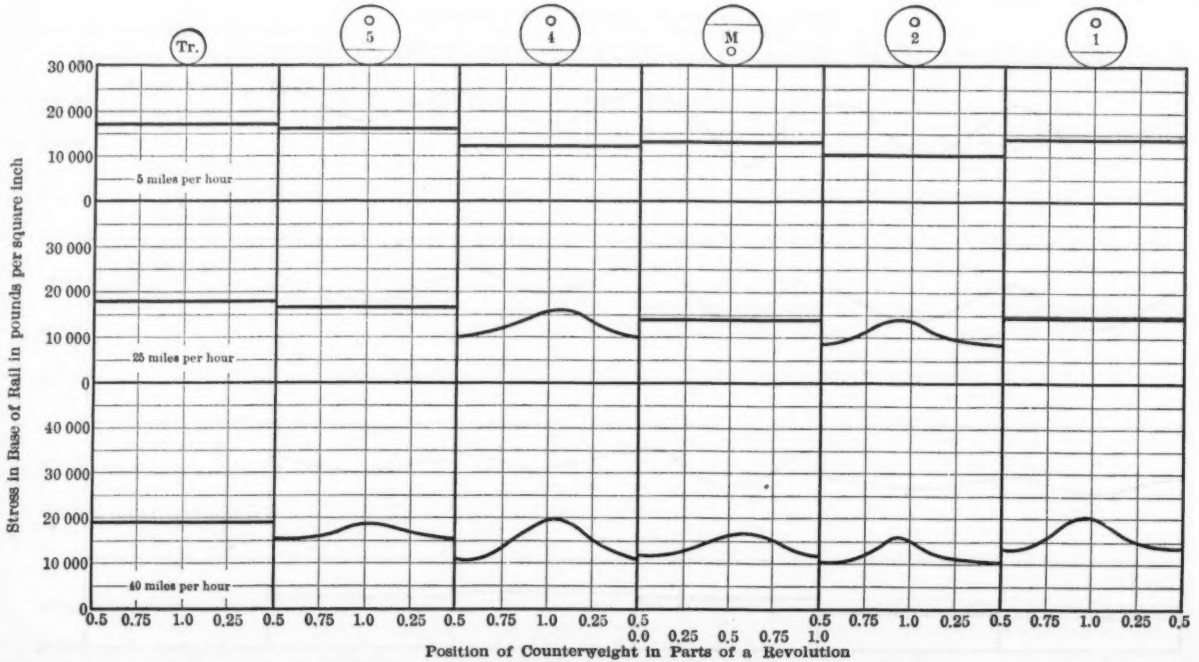


FIG. 28.—CURVES OF AVERAGE STRESS IN RAIL ON STRAIGHT TRACK THROUGHOUT REVOLUTION OF DRIVER, SERIES 5401–5408, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

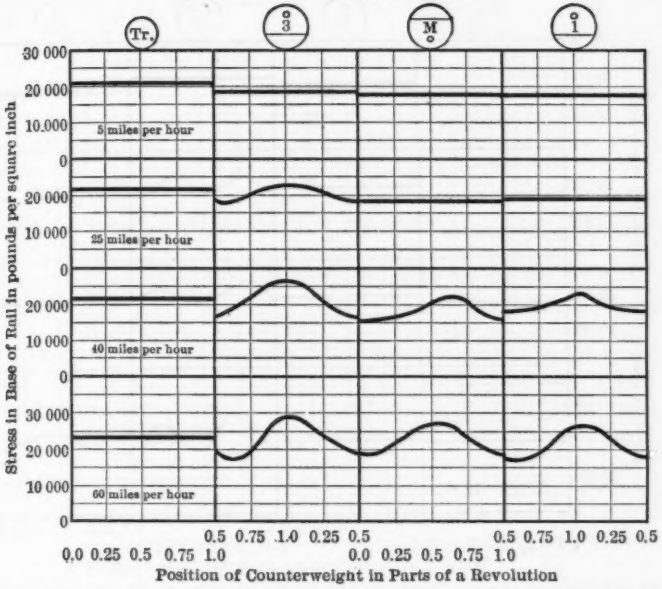


FIG. 29.—CURVES OF AVERAGE STRESS IN RAIL ON STRAIGHT TRACK THROUGHOUT REVOLUTION OF DRIVER, SERIES 5450–5458, PACIFIC TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

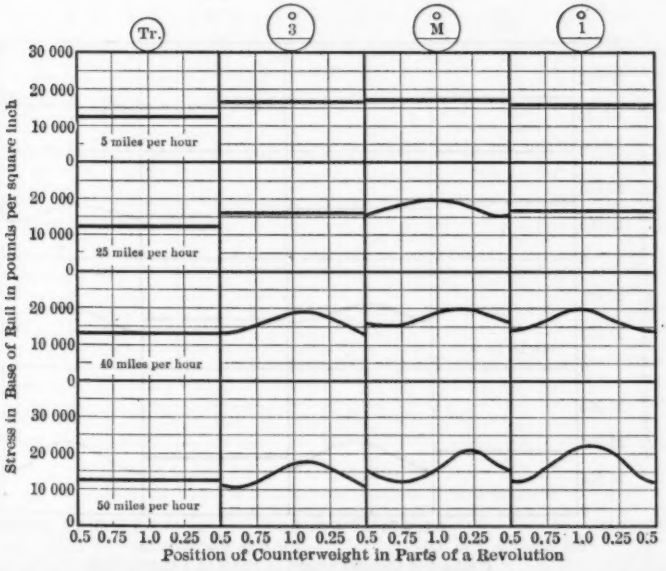


FIG. 30.—CURVES OF AVERAGE STRESS IN RAIL ON STRAIGHT TRACK THROUGHOUT REVOLUTION OF DRIVER, SERIES 5441–5449, LIGHT PRAIRIE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

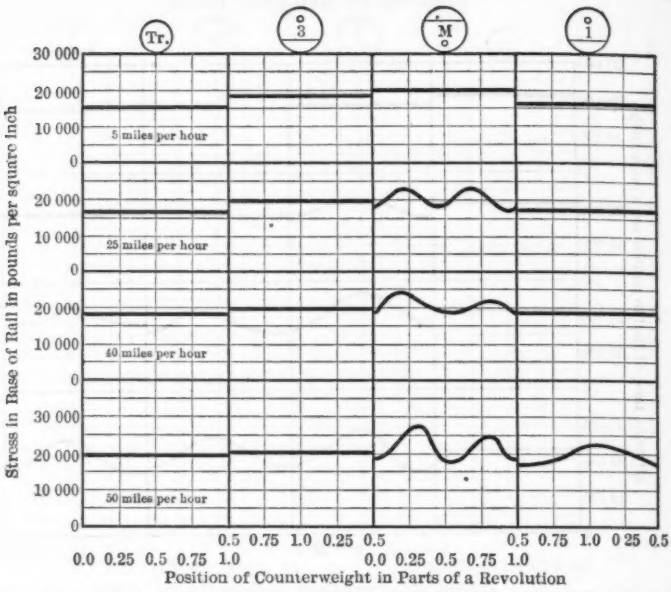


FIG. 31.—CURVES OF AVERAGE STRESS IN RAIL ON STRAIGHT TRACK THROUGHOUT REVOLUTION OF DRIVER, SERIES 5434–5440, BALANCED COMPOUND PRAIRIE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

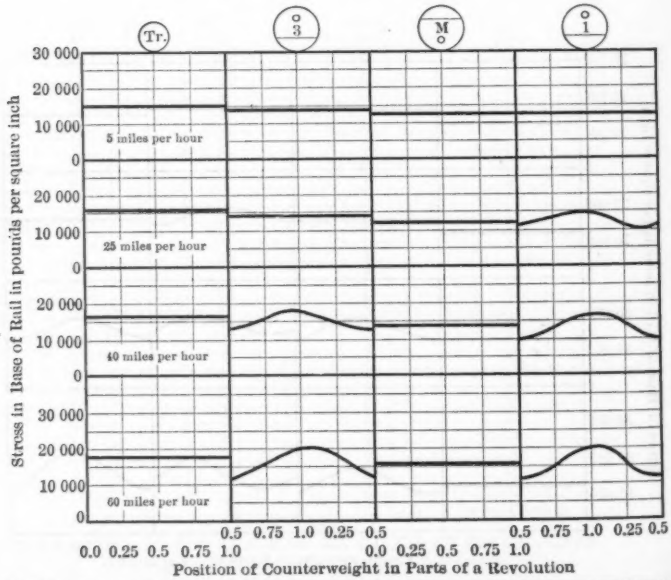


FIG. 32.—CURVES OF AVERAGE STRESS IN RAIL ON STRAIGHT TRACK THROUGHOUT REVOLUTION OF DRIVER, SERIES 5136–5149, PACIFIC TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

effect of the plane of motion of the main rod and other outside rotating parts not being coincident with the plane of motion of the counterweight and wheel and rail, whereas, the opposite condition may be shown by the results. It is evident from the discussion of this subject in the second progress report that a proper consideration of the relative positions of the plane of the rotating parts and of the wheel will account for this seeming contradiction.

Two locomotives of those used have characteristics which differ from those previously described. The Pacific type locomotive of the Delaware, Lackawanna and Western Railroad has no appreciable counterbalance effect under the main driver (see Fig. 25 and Fig. 32); this may be expected from the data of the counterbalance of Table 8. The Balanced Compound Prairie type locomotive of the Atchison, Topeka and Santa Fe Railway has four connecting rods, and there is fair counterbalance when these rods are at the high and low positions. (See Fig. 24 and Fig. 31.) When the counterweight is half way between high and low position, there is an unbalanced effect; at 50 miles per hour, the additional stress due to this effect is 6 200 lb. per sq. in. for one position and 3 400 lb. per sq. in. for the other. The increased stress in rail due to counterbalance in this locomotive is, therefore, greater than that found for the other locomotives except the Light Santa Fe type, and this, notwithstanding it may have been expected that the balanced compound would give low counterbalance effects.

In the case of the Light Santa Fe type locomotive the smallness of the diameter of the wheel prevented the application of sufficient counterweight to the main driver to give a proper counterbalance, even though bobs were used. For the Heavy Santa Fe type locomotive, the counterweight on the main driver was nearly as heavy as that required when all the rotating parts are considered to be in the same plane as the wheel, but the high stress for the counterweight up, shows the need for a revision of such calculations, as do the other locomotives.

A study of the values of the stresses attributable to counterbalance, as given in the tables and diagrams, indicates that these stresses are not unreasonably high, with the exception of the Light Santa Fe type locomotive.

In the Pacific type locomotive of the Atchison, Topeka and Santa Fe Railway, the additional stress in rail attributable to counterbalance at a speed of 60 miles per hour is 5 500 lb. per sq. in. under the third driver, which is 29% of the average stress found at 5 miles per hour. The highest stress under the main driver occurs when the counterweight is up.

In the Light Prairie type locomotive, the highest counterbalance stress at 50 miles per hour is under the first driver, 5 200 lb. per sq. in., which is 33% of the average stress under this driver at 5 miles per hour.

In the Mountain type locomotive, the highest counterbalance stress at 60 miles per hour is under the main driver when the counterweight is up, 2 900 lb. per sq. in., which is 21% of the average stress under this driver at 5 miles per hour. It is well to note that this locomotive rode smoothly and evenly. The counterbalance stresses were unusually small.

The Light Santa Fe type locomotive (an old design, the first of this type), gives relatively high counterbalance effects. Although the driver loads are

small, only about 83% of the wheel loads of the other locomotives tested, the additional stress in rail under the first driver, due to counterbalance, is 7 700 lb. per sq. in., and that under the main driver, 9 200 lb. per sq. in. These counterbalance effects are 64% and 114%, respectively, of the average stresses under these drivers at 5 miles per hour, unusually large counterbalance effects. The observed stresses themselves are not exceptionally high, the rail being relatively heavy for the wheel load of this locomotive, but the stresses indicate very poor counterbalancing. This condition of counterbalance is doubtless due to inability to add adequate counterweight to such small drivers.

For the Heavy Santa Fe type locomotive, the greatest increase in stress in rail due to counterbalance was under the fourth driver, 5 300 lb. per sq. in. at 40 miles per hour, which is 43% of the average stress in rail under this driver at 5 miles per hour. The next greatest counterbalance stress was under the first driver. The greatest stress under the main driver occurred when the counterweight was up. This locomotive rode smoothly and easily even at 40 miles per hour.

For the Pacific type locomotive of the Delaware, Lackawanna and Western Railroad, the greatest increase in stress in rail due to counterbalance was 4 300 lb. per sq. in. under the first driver at 60 miles per hour, which is 35% of the average stress in rail under this driver at 5 miles per hour. The third driver gave about as great counterbalance stress as this at 60 miles per hour. Under the main driver, the counterbalance effect was so small as not to give a definite deviation from a straight-line representation of the tests. This locomotive was, in general, a smooth riding locomotive.

The Mikado type locomotive of the Delaware, Lackawanna and Western Railroad was not run faster than 25 miles per hour on straight track; at the three speeds used, the plotted points showed no counterbalance effect.

In general, for all the locomotives except the Light Santa Fe type, the stresses attributable to counterbalance at the highest speeds run, which in some cases were greater than the scheduled maximum speed allowed for the locomotive, are less than 5 000 lb. per sq. in. and, in most cases, considerably less than this. The counterbalance effect at the highest speeds used may be said generally to be not more than 30 or 40% of the average stress in rail at 5 miles per hour; in some few cases, it runs higher; but, in most cases, it is lower. A counterbalance effect of 30% will not be considered to be excessive, and the only question is whether the scheduled speeds are likely to be exceeded very much. It is seen that an increase of speed of 25% will increase the counterbalance effect by 56% of itself, and an increase of speed of 41% will double the counterbalance effect. It is evident that great care must be exercised to prevent the scheduled limits of speed being exceeded very much if the stresses due to counterbalance are to be kept within moderate bounds. It is also clear that every care should be taken in the design, construction, and maintenance of the proper counterbalance, and checks should be exercised to insure that the conditions of counterbalance are known with certainty, as it has frequently happened that the counterbalance effect has varied from the reported or expected condition. Attention should be given to the main driver and a

careful determination made of the effect of the outside rotating parts not being in the plane of the wheel and counterweight.

In the locomotive for which the weights and positions of the rotating parts seem to be accurately known, the value of the additional stress due to counterbalance, calculated by the use of the calculated vertical dynamic augment, agrees fairly closely with the observed counterbalance stress as found by the method already described. For the main driver, the difficulties are increased, but accurate knowledge of the position of the several outside rotating parts with reference to the plane of the driver itself should permit a fairly close determination of the counterbalance stress under this driver. Any method of design or improvement in materials that will help to keep small the counterbalance effect should be welcomed by both the Mechanical and the Engineering Departments.

8.—Effect of Speed and Combined Effect of Speed and Counterbalance.—If the stresses in rail found in tests made throughout the revolution of a driver are averaged, the effect of counterbalance may be considered to be eliminated. The ratio of this average of the stresses throughout the revolution for a given speed to the average stress at 5 miles per hour may be taken to represent the effect of speed alone. In general, the tests show that the effect of speed on stress in rail under the drivers in all except the Light Santa Fe type locomotive was not large. The speed effect under the leading truck wheels, trailer, and tender wheels is also of interest. The average values of the stress in rail throughout the revolution of the drivers and of the other wheels may be found in the tables and diagrams referred to previously.

In Table 19, the principal values of the percentages of increase in stress in rail under the various wheels of the locomotives, due to increase in speed above 5 miles per hour, are recorded, together with values of the increase due to combined speed and counterbalance effect under the drivers. The general values given represent increases that are common to several wheels, and the maximum value relates to the highest of the group of wheels.

With the exception of the Light Prairie type and the Light Santa Fe type, it will be seen that the general increase in stress in rail under the drivers due to speed alone in changing from 5 miles per hour to the highest speed used in the tests ranges from 15 to 27%, which may be called a moderate and allowable speed effect. With the same exceptions, the maximum speed effect under any one driver ranges from 24 to 30 per cent. For these locomotives and these tracks, therefore, the increase in stress in rail due only to the change in speed noted may be said generally to be less than 35 per cent.

For the combined effect of speed and counterbalance for the same locomotives for the maximum speed used, the range of general total increase in stress in rail is 30 to 53%, and the maximum increase under any driver ranges from 43 to 80 per cent. The stresses in rail at the maximum range from 20 000 to 29 000 lb. per sq. in., in the three sections of rails used. It should be borne in mind that these stresses are the averages of the stresses in the two edges of the base of rail, and, also, that they do not take into account the variations from the average, which are bound to occur, nor the effect of speeds higher than those

normally used. It should be remembered also that the track was substantial track in good condition and that the locomotives were of good design and in good working order.

TABLE 19.—PRINCIPAL VALUES OF INCREASE IN STRESS IN RAIL AT A GIVEN SPEED OVER THE STRESS AT 5 MILES PER HOUR.

The increase is given in percentage of the stress at 5 miles per hour.

Type of locomotive.	Drivers.				Trailer.	Tender Wheels.	
	Speed Alone.		Combined Speed and Counterbalance.				
	General value.	Maximum under one driver.	General value.	Maximum under one driver.		General value.	Maximum under one wheel.
Atchison, Topeka and Santa Fe Railway:							
Pacific, 60 miles per hour	27	30	53	53	15	75	93
Balanced Compound Prairie type, 50 miles per hour	15	26	30	43	29	50	87
Mountain, 60 miles per hour	20, 20	30	35	50, 55	10	40, 60	110
Light Santa Fe, 45 miles per hour	40	77	105	190	10	25	35
Heavy Santa Fe, 40 miles per hour	18, 18, 18	28	50	80	15	75, 70, 50	110
Delaware, Lackawanna and Western Railroad:							
Pacific, 60 miles per hour	20	24	42	59	14	45	57

The increase in stress due to speed alone for the Light Prairie type locomotive by the method of analysis used is very small, much smaller than for the other types of locomotives, and varies from nothing to 5 per cent. No explanation for this is offered. The combined effect of speed and counterbalance (by the method of analysis used, most of which is the effect of counterbalance) is 40% for the first driver and about 20% for the other two drivers.

Although the stresses found with the Light Santa Fe type locomotive at the maximum speed were not exceptionally high, the driver loads being light, and their spacing close, the percentage of increase in stress in rail under the drivers for both speed and combined speed and counterbalance was markedly high. The increase in stress due to speed alone was 47% under the fourth driver and 77% under the main driver; the increase in stress due to combined effect of speed and counterbalance under the several drivers was 87, 74, 190, 97, and 73%, respectively. It may be added that this locomotive was built to be used with a rail lighter than the 90-lb. S. F. rail section that was used in the track tested.

In none of the locomotives tested was the effect of the counterbalance of the drivers transmitted to the trailer in sufficient amount to be brought out in the stress-position of counterweight curves, as was the case in some of the locomotives discussed in the second progress report. The increase in stress under the trailer, due to speed, was, in general, less than that under the

drivers, although the value for the Balanced Compound Prairie type locomotive (27%) was as great as under any driver. In general, the increase noted at the highest speed was from 10 to 15 per cent.

For the locomotives with a two-wheel leading truck, the proportionate increase of stress in rail under the truck due to speed is generally somewhat less than that found under the drivers. The increase in stress at the highest speed is about 20 per cent. For the locomotives with four-wheel leading trucks, the increase under the front wheel is about 35% for the two Pacific type locomotives and 20% for the Mountain type. The increase under the rear wheel is much greater proportionally than that under the front wheel, although as the stress under the rear wheel at static load is very small, because of the proximity of the first driver, the stress at the highest speed is not large, being generally less than that under the front wheel.

The effect of speed on stresses in rail under the wheels of the tender varies with the type of locomotive; the percentage of increase is generally considerably larger than that under the drivers. In the Pacific type of the Atchison, Topeka and Santa Fe Railway and the Heavy Santa Fe type, the increase of stress in rail at the highest speed is approximately 75 per cent. In the other locomotives, the range is from 40 to 60 per cent. The stresses at both 5 miles per hour and at the maximum speed are smaller than the corresponding stresses under the drivers. How much of this large proportional increase in stress is due to the smallness of the wheel diameter and how much to the relatively small load on the wheels or to the way in which the truck is directed and even to the condition of the springs is not known.

9.—General Discussion of Tests on Straight Track.—With the exception of those found with the Light Santa Fe type locomotive, the tests indicate no abnormal stresses in rail under any of the wheels at any speed. The locomotive design is generally within the requirements of what has been considered acceptable practice by Mechanical Departments. With a single exception, the locomotives rode smoothly, showing that there was probably no abnormal strain on the frame of the locomotive. As there are differences in the division of the loads and in the counterbalancing and in the spacing of wheels, some consideration and comparison of the stresses developed in the rail under the various wheels of the locomotives may be useful.

In most cases, the loads on the drivers are moderately heavy, about 30 000 lb. on a driver, reaching 33 000 lb. in the case of the Pacific type locomotive of the Delaware, Lackawanna and Western Railroad. The load on the trailers differs considerably. The leading truck wheels, either two-wheel or four-wheel trucks, each carries about one-half as much load as a driver. Part of the tenders have four-wheel trucks and part six-wheel; the wheel loads of the latter are about the same as those of the former.

The spacing of the drivers is, of course, greater for the Pacific and Prairie types than for the others; the greater spacing results in a somewhat greater proportional effect of the weight on the stresses developed under the drivers. On the other hand, the shorter spacing of the drivers in the Light Santa Fe type locomotive decreases the stresses.

Leaving out of consideration the Light Santa Fe type locomotive of the Atchison, Topeka and Santa Fe Railway and the Pacific type locomotive of the Delaware, Lackawanna and Western Railroad, the latter having heavier driver loads, it is seen that the several locomotives develop stresses under the drivers at 5 miles per hour, that do not differ greatly if allowance is made for differences in rail section. Also, there is generally no great difference in stresses for any group of drivers, the stress under the first and last driver naturally being somewhat greater than under the others, as is indicated in Fig. 6* of the first progress report. In the Heavy Santa Fe type locomotive, the stresses vary more, ranging from 10 100 to 15 900 lb. per sq. in. The latter stress, which is under the fifth driver, is even greater than would be expected from analytical considerations (the stress under the trailer being correspondingly less), indicating that the division of load is not that given in the diagram of locomotive loading.

In general, the effect of speed alone (omitting the effect of counterbalance) is as small as could be expected. Even the 27% increase at 60 miles per hour found with the Pacific type locomotive of the Atchison, Topeka and Santa Fe Railway seems moderate. The increase of 18% at 40 miles per hour found with the Heavy Santa Fe type locomotive, compared with the Pacific type locomotive, is directly proportional to the increase of speed, so that this freight locomotive appears to produce about the same relative increase as the passenger locomotive, if the conditions of the track are considered to be the same. It will be seen that the increase at the drivers that show the maximum speed effect, gives somewhat the same comparison, although the increase for one driver of the Santa Fe type (28%) is relatively larger.

The percentage of increase in stress in rail due to counterbalance at the highest speeds used with the different locomotives (see Table 19) generally shows an average for the several drivers that is about the same as the general value for the increase in speed alone. In the case of the Heavy Santa Fe type locomotive, the increase due to counterbalance is greater proportionally than for the other types; this may be due to a poorer opportunity for counterbalancing. The maximum increase due to counterbalance for some one driver of a locomotive is generally considerably greater than the average for the drivers; this shows a need for more careful study of the counterbalancing problem. Further, attention should be called to the discrepancy between the condition of counterbalance in the main driver usually assumed in design and the condition found in the test. This discrepancy was found in every locomotive tested; the difference is explainable at least partly by the neglect to recognize the effect produced by the outside rotating parts not being in the same plane as that of the wheel and rail. It would appear also that the counterbalancing of other drivers of some of the locomotives might be improved. It should be noted also that for speeds still higher than those used the effect of counterbalance may be expected to increase as the square of the velocity.

The general values given in Table 19 for the general increase in stress at the maximum speed used due to combined speed and counterbalance (except

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1209.

for the Light Santa Fe type as before) range from 30 to 53 per cent. It would appear that these values may be considered to be representative of the increase which may be expected with locomotives of fair design in good condition and with track in good condition. At an individual driver on the Heavy Santa Fe type locomotive, the increase amounts to 80 per cent. If speeds still higher than those used in the tests are likely to be run even occasionally, the fact that the stress in rail will be increased more rapidly than as the first power of the speeds, should receive consideration in judging of the effect on rail and track.

With the exception of the Light Santa Fe type locomotive, the nominal load on the trailer ranges from 72 to 109% of the average load on the drivers. It will be recalled that analytical considerations show that with the trailer well away from the rear driver the stress in rail will be greater accordingly than that under a driver, and that to give an equal stress under static conditions the load on the trailer should be about 75 or 80% of that on a driver. (For an illustration of this effect see Fig. 6 of first progress report previously mentioned.) The observed stresses bear out the relations shown by the analysis except in the case of the Mountain type and the Heavy Santa Fe type locomotive, with which the observed stresses under the trailer, at 5 miles per hour, were 15 to 20% lower than the calculated stresses and the division of load among drivers and trailer was evidently not as reported on the locomotive diagrams, the higher stresses under the drivers indicating that the drivers were taking a greater share of the load than was planned and that the trailer was taking less. The stresses under the trailer of the locomotives of the Delaware, Lackawanna and Western Railroad agreed closely with the calculated values. In four of the types of locomotives used in the tests, the stress under the trailer at 5 miles per hour is greater than the average stress under the drivers. The excess of stress under the trailer over the average stress under the drivers, at a speed of 5 miles per hour, was, as follows: Pacific type, 15%; Mountain type, 22%; Heavy Santa Fe type, 31%; and Pacific type of the Delaware, Lackawanna and Western Railroad, 18 per cent. The increase in stress under the trailer due to speed is generally less than that under the drivers due to speed alone, and when the effect of counterbalance is considered much or all of the discrepancy between the stresses under the trailer and the drivers may disappear, and there will also be the lesser effect of the depression of the track by a single load. It should be recalled that in the tests recorded in the second progress report, there was little difference in effect of speed between trailer and drivers.

For the purpose of comparing the effect of two-wheel and four-wheel leading trucks, the Pacific and Mountain type locomotives, both having four-wheel trucks, may be compared with the Prairie and the Heavy Santa Fe type locomotives having two-wheel trucks. One wheel of the four-wheel truck carries about one-half as much load as a driver, and in the two-wheel truck a wheel also carries about one-half as much as a driver. In the four-wheel truck, the leading wheel develops 50 to 60% as much stress in the rail at 5 miles per hour as a driver; a second wheel produces 25 to 40% as much as a driver, the smaller values under this wheel being due to the proximity of the leading

driver. With the two-wheel truck, the stress in rail under the wheel is 50 to 60% of the stress under a driver. The highest stresses under a truck wheel are found under the leading wheel of the Pacific type locomotive (13 800 lb. per sq. in., at 60 miles per hour) and under the wheel of the Balanced Compound Prairie type locomotive (14 800 lb. per sq. in., at 50 miles per hour). these are not unduly high stresses. Those found with the other types are smaller, but all of them are great enough to insure sufficient pressure on the rail and sufficient depression of rail relative to that under the other wheels to give stable conditions. It would appear that, as far as straight track is concerned, either two-wheel or four-wheel trucks will give stresses in rail which should be considered acceptable; this conclusion allows latitude in the design of the locomotive.

The following locomotives have six-wheel trucks under the tender, Pacific, Mountain, and Heavy Santa Fe types of the Atchison, Topeka and Santa Fe Railway. All the other locomotives tested have four-wheel tender trucks. The weights on the individual wheel when the tender is fully loaded do not differ greatly, ranging from 19 000 to 23 500 lb., except in the Light Prairie type. At the time of the test, the effort was made not to have the loads vary much, water being taken frequently; but the usual load was less than full load. The stresses in rail on straight track at a speed of 5 miles per hour were generally from 8 000 to 10 000 lb. per sq. in., being about two-thirds of those under the drivers. As already noted, the average increase at the highest speed is 40 to 75%, the resulting stress at the highest speed is usually considerably less than the average stress developed under the drivers, although in the Heavy Santa Fe type locomotive the stress under the wheels of the tender at the highest speed reached (about 16 000 lb. per sq. in.) almost as great a value as the average stress under the drivers. The great variation in the effect of speed under the wheels of the tenders of the various locomotives agrees with the results noted in the second progress report. It seems that there must be considerable difference in the design and construction of tenders or in their maintenance.

The placing of a double trailer (two trailer axles) under a Heavy Santa Fe type locomotive for the purpose of learning experimentally the effect of this form of construction in changing the stresses in rail under the trailers and under the drivers was a novel undertaking that gave results of much interest. Although the double trailer was put in one of the regular locomotives and thus the design did not have all the proportions which would be desired in a new design, it gave very successful results and decreased the stresses under both drivers and trailer, at the same time working smoothly and satisfactorily. The introduction of the double trailer brings a marked change in the distribution of stress under the several wheels of the locomotive. The stress under the trailer in the 90-lb. S. F. rail is changed from 16 900 lb. per sq. in. in Locomotive No. 3813 to 9 000 lb. per sq. in. under each trailer in Locomotive No. 3829. Considerable load has evidently been removed from the last three drivers, as the stress, at 5 miles per hour, was decreased 40% under the fifth driver. With these changes has come an increase of 60% in the stress under

the wheel of the front truck, although the resulting stress is not especially high on the straight track. Under the trailers, the effect of speed is to increase the stress 25%, in the change from 5 to 40 miles per hour. Even at 40 miles per hour, the stress in the 90-lb. S. F. rail under the trailers is only 11 300 lb. per sq. in. and the stresses under the drivers (not including the effect of counterbalance) are only about 13 700 lb. per sq. in. The stresses indicate a smoothly running locomotive on straight track. It develops, however, that the presence of two trailer axles does not give adequate room or load to permit the addition of booster equipment, which is now desired with this type of locomotive.

Another locomotive not included in the preceding discussion is the Light Santa Fe type. It differs from most of the other locomotives in having lighter driver loads and in having nominally a very light load on the trailer. The spacing of the drivers is somewhat closer and the trailer is only 7 ft. away from the fifth driver. All these differences favor the development of small stress in the rail. At 5 miles per hour, the stress under the first driver is more than 50% greater than that under the third, fourth, and fifth drivers. The stress under the trailer is double that calculated from the weight given on the locomotive diagram. It is evident that the weights are not distributed as given in the diagram; the average of the observed stresses under drivers and trailer, however, closely agrees with the average of the calculated values. This locomotive is marked by large increase in stress in rail under the drivers, due to both speed and counterbalance. A change in speed from 5 to 45 miles per hour increases the stress in rail under the main driver 77% for speed effect alone, and 113% for counterbalance alone, the combined effect for speed and counterbalance being an increase in stress of 190 per cent. Due to the lighter loads and close spacing of the wheels, the stresses in rail, even at 35 miles per hour, are less than 20 000 lb. per sq. in. in the 90-lb. S. F. rail. At 45 miles per hour, higher stress is found. This is a rough riding locomotive. Its behavior in causing lateral bending of the rail, which is discussed subsequently, and the high increases in stress with increased speed are undesirable features, although it should be noted that the first of these locomotives were built in 1902 and were the first of the 2-10-2 Class built by any road.

The calculation of stresses in rail by the analytical method given in the first progress report in general gives results which agree closely with the observed stresses at 5 miles per hour. The calculated stresses are given in Tables 10 to 18, inclusive. In some cases, the stress under one wheel is higher and that under another is lower, but the average of the stresses under all the wheels checks closely, indicating slight differences between the actual and the nominal distribution of load among the drivers. In one or two cases, the distribution of load evidently differs from the nominal values. Some change in distribution is sometimes seen when the speed is increased. This agreement of values should give added confidence in the accuracy of the analytical method.

10.—Stresses at the Two Sides of the Locomotive.—In view of the experience with one of the locomotives reported in the second progress report, it was thought desirable to make a study to learn whether the stresses in the

rail on one side of the locomotive differ from those on the other side. Tests with three locomotives were made on straight track with four instruments on the south rail and four instruments directly opposite on the north rail. During the first day, the locomotive was run eastward at the several speeds, and during the second day, its direction was reversed and runs were made over the same track at the same speeds. The instruments were left on the rail over night and other conditions remained the same. The four instruments on the south rail of the eastward runs were averaged with the four instruments of the north rail

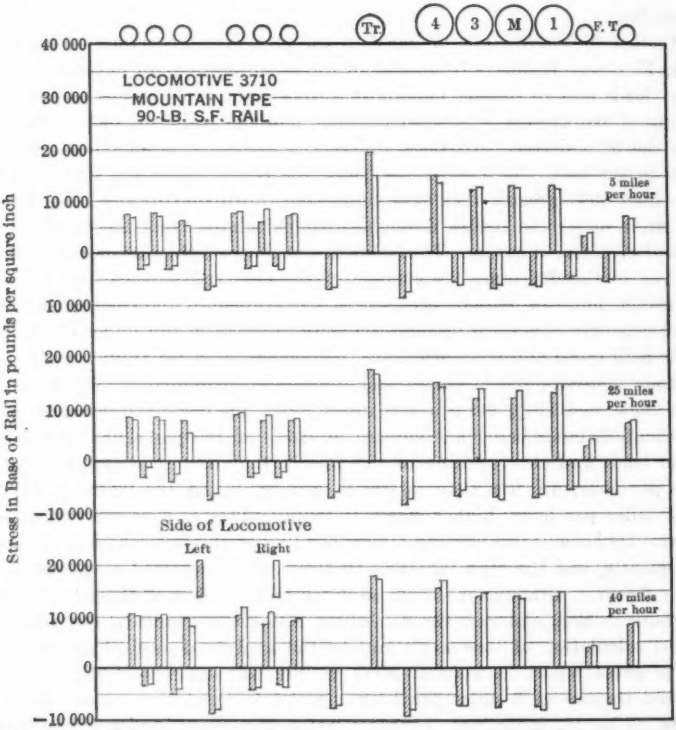


FIG. 33.—MEAN STRESS IN RAIL AT THE RIGHT AND LEFT SIDES OF THE MOUNTAIN TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

of the westward runs, giving the stresses for the right side of the locomotive. Similarly, the instruments at the left side of the locomotives were averaged for the two directions. It would seem that the tests obviated possible differences due to variations in instruments and in conditions of the track on the two sides, and that, therefore, the results of the tests for the two sides of a locomotive may properly be compared.

In Fig. 33, the stresses under the sides of the Mountain type locomotive are given. It is seen that the stresses under the left side are nearly the same

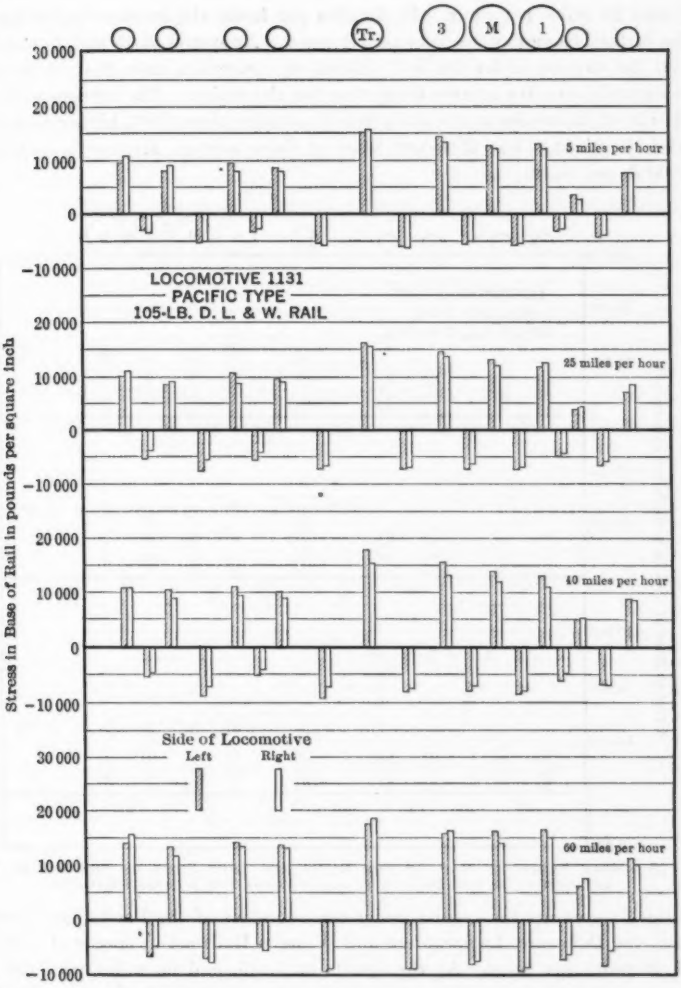


FIG. 34.—MEAN STRESS IN RAIL AT THE RIGHT AND LEFT SIDES OF THE PACIFIC TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

as those under the right side. The only variation of any amount is under the trailer at 5 miles per hour.

In Fig. 35, the stresses under the two sides of the Mikado type locomotive of the Delaware, Lackawanna and Western Railroad are given for speeds of 5, 15, and 25 miles per hour. At 5 miles per hour, the stresses under the two sides of the locomotive are very nearly equal. At speeds of 15 and 25 miles per hour, the stresses under the left drivers are somewhat more than those under the right drivers, the reverse being true for the trailer. The stresses under the left side of the tender at the three speeds average about 10% higher than those under the right side. However, none of these average stresses is as high as 15 000 lb. per sq. in.

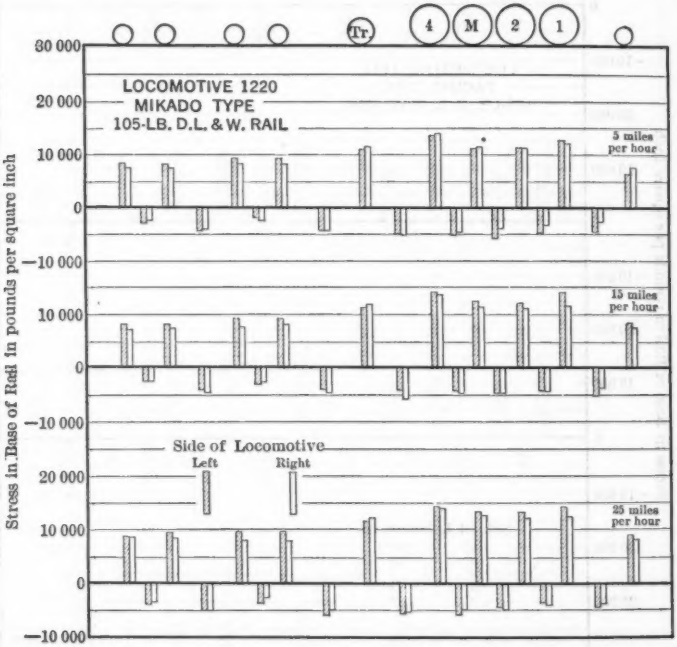


FIG. 35.—MEAN STRESS IN RAIL AT THE RIGHT AND LEFT SIDES OF THE MIKADO TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

Fig. 34 presents the results under the two sides of the Pacific type locomotive of the Delaware, Lackawanna and Western Railroad at speeds of 5, 25, 40, and 60 miles per hour. At the speeds of 5, 25, and 60 miles per hour, the stresses under the two sides of the locomotive and tender average approximately the same, but at the speed of 40 miles per hour the stresses under the left side averaged about 10% more than those under the right side. None of the average stresses is greater than 18 000 lb. per sq. in.; in no case does the stress under the right and left wheel of an axle differ more than 2 000 lb. per sq. in. and, generally, the difference is much less.

It may be said then that for both types of locomotive the stresses in rail under the two sides of the locomotive were found to differ only slightly. It appears that it more frequently happens that the stress under the left wheel of an axle is somewhat greater than that under the right wheel than the reverse; about two times out of three the stress at the left side is greater than that at the right. There seems to be no noticeable difference in this relation at the lower and the higher speeds, so that the variation found with these engines is not caused by speed or counterbalance. Besides, the tender wheels show the same general variation as the wheels of the locomotive. It seems probable then, that, in these locomotives, the variations between the stress in the rail at the left and the right sides are no more than are to be expected with any well-built machine.

11.—Stresses at the Two Edges of the Base of Rail.—In the two preceding progress reports, reference was made to the frequent and common lateral bending of the rail in straight track, and it was shown that the stresses due to such bending attain considerable magnitude. The data of the tests described in this report give added information on the subject.

In Fig. 36 is plotted the ratio between the stress at the outside edge of rail and the average of the stresses at the two edges (mean stress in base of rail) for each observation under the first pair of drivers of the Light Santa Fe type locomotive and at three speeds. The ratio of the stress at the inside edge of rail to the mean stress in base of rail may be found by subtracting from 2 the ratio given in the diagrams. It should be noted also that the ratio of stress at the outside edge to that at the inside edge may be found by dividing one of these ratios by the other. The value of the stress which corresponds to an observation that gives a ratio of 1.00 is the same as the average stress for the given wheel as it is recorded in the diagrams and tables of general stress values. A mean line drawn through the average of the ratios is also shown. Fig. 36 also gives the stresses at the inside edge and at the outside edge for the same drivers. Fig. 37 gives similar data for the first pair of drivers of the Heavy Santa Fe type locomotive. Table 20 gives values of the average ratio of the stress at the outer edge of rail to the average stress at the two edges for some of the wheels of several locomotives.

A study of these diagrams (confirmed by an examination of the diagrams of the other wheels, which are not reproduced) shows considerable variation between the stresses at the two edges. The stress at the outer edge of the base of rail is quite frequently 33% or more greater than the mean stress in base of rail, in a few cases reaching an excess of 50% or more, and this holds true throughout the revolution of the driver. A stress at the inner edge 33% greater than the mean stress and even more was found, although less frequently. This means that in these cases the stress at one edge of the base is twice as much as that at the other, or more.

Another way of considering the range in stress at the two sides of the rail, which seems useful in judging of the amount and effect of lateral stresses, is that of noting, for any wheel, the range of stress on each side of the mean line in a diagram, such as Fig. 36 or Fig. 37. It will be seen that usually the

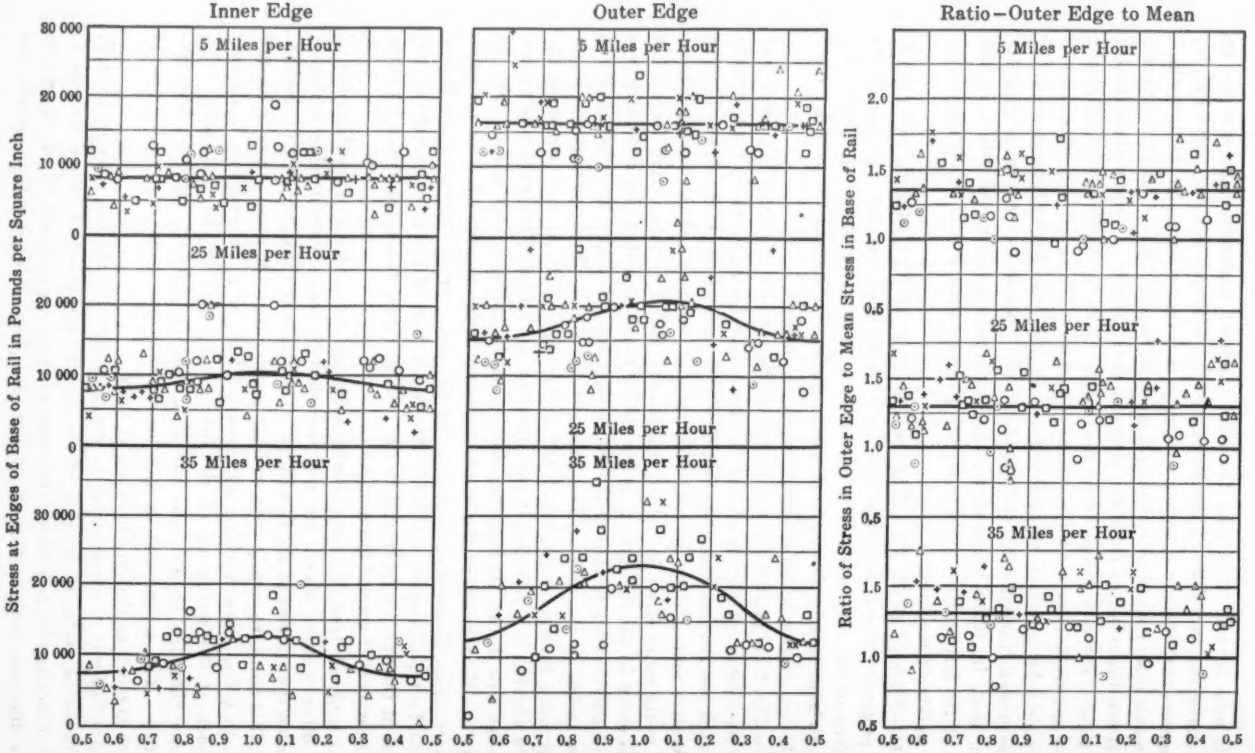


FIG. 36.—STRESSES AT INNER EDGE, OUTER EDGE, AND RATIO OF STRESS IN OUTER EDGE TO MEAN STRESS IN BASE OF RAIL THROUGHOUT THE REVOLUTION OF THE FIRST DRIVER OF THE LIGHT SANTA FE TYPE LOCOMOTIVE ON STRAIGHT TRACK.

FIG. 36.—STRESSES AT INNER EDGE, OUTER EDGE, AND RATIO OF STRESS IN OUTER EDGE TO MEAN STRESS IN BASE OF RAIL THROUGHOUT THE REVOLUTION OF THE FIRST DRIVER OF THE LIGHT SANTA FE TYPE LOCOMOTIVE ON STRAIGHT TRACK.

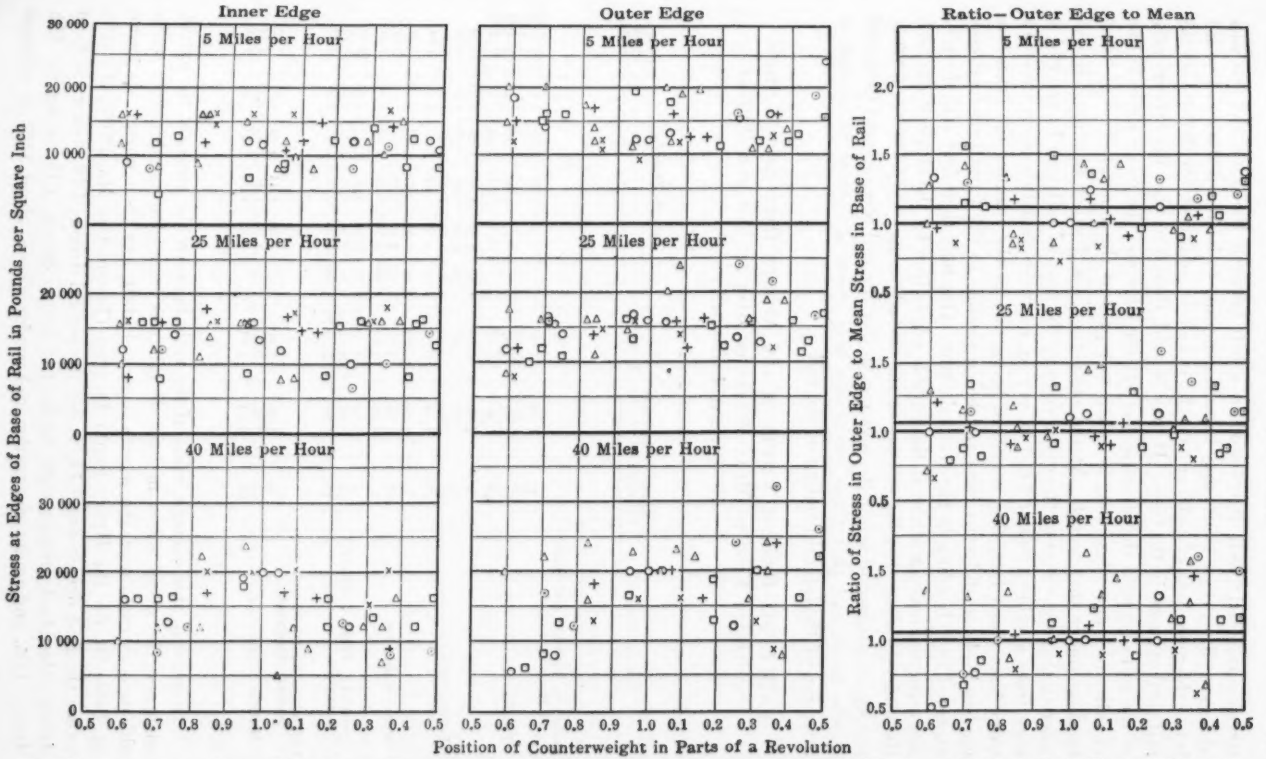


FIG. 37.—STRESSES AT INNER EDGE, OUTER EDGE, AND RATIO OF STRESS IN OUTER EDGE TO MEAN STRESS IN BASE OF RAIL THROUGHOUT THE REVOLUTION OF THE FIRST DRIVER OF THE HEAVY SANTA FE TYPE LOCOMOTIVE ON STRAIGHT TRACK.

stresses are fairly well distributed in a belt between limits on the two sides of a mean line; these limits may be said generally to be at a distance above and below the mean line of 4 000 or 5 000 lb. per sq. in. of stress, for the stresses and rail sections used in the tests. It may be said, therefore, that the average value of the ratio between the stress at the outside edge and the mean stress in base of rail under any wheel may be taken to represent the general effect of lateral bending under that wheel. At the same time, the further fact must be considered, that the stress at the given edge of the base of rail may be expected to vary from a value 4 000 or 5 000 lb. per sq. in. more than this mean stress to a value that much less, this variation being due to the manner the locomotive applies itself on different runs and being similar to the variations in stresses expected from bending in the vertical plane. One may then deal with the average ratio and average stress at the outside edge given by the observations, with the understanding that the stresses at the two edges may vary 4 000 or 5 000 lb. per sq. in. above and below the mean value for the rail sections and loads used in the tests, and that these variations may cause a change in amount or direction of the lateral bending of the rail.

TABLE 20.—General Values of the Average Ratio of the Stress at Outer Edge to Mean Stress in Base of Rail for Wheels of Several Locomotives.

Locomotive.	Speed, in miles per hour.	Average of tender wheels.	Trailer.	Driver Number.					Front truck wheels.
				5	4	3	2	1	
Atchison, Topeka and Santa Fe Railway:									
Pacific	5	1.08	1.05	....	. . .	1.17	1.14	1.12	1.08 1.01
	25	1.06	1.04	....	....	1.14	1.12	1.12	0.96 1.05
	40	1.03	1.07	....	....	1.08	1.10	1.12	1.05 1.05
	60	0.99	1.07	....	....	1.08	1.13	1.17	1.08 1.01
Light Santa Fe	5	1.00	0.96	1.10	1.10	1.01	1.26	1.33	1.05
	25	1.01	0.97	0.99	1.06	1.05	1.29	1.31	1.04
	35	1.00	0.99	0.96	1.09	1.05	1.24	1.31	1.00
	45	1.04	0.94	1.00	1.02	1.07	1.27	1.20	1.04
Heavy Santa Fe	5	1.13	1.03	1.08	1.18	0.97	1.16	1.12	1.01
	25	1.07	1.01	1.05	1.15	0.93	1.10	1.06	0.97
	40	1.00	1.05	1.03	1.13	0.89	1.10	1.06	0.97
Double Trailer, Heavy Santa Fe	5	1.03	1.02 0.98	0.96	0.98	0.97	1.00	1.05	0.97
	25	1.05	1.04 1.01	0.97	0.98	0.94	1.01	1.01	0.96
	40	1.05	1.05 1.05	1.08	1.05	0.99	1.05	1.07	1.04
Delaware, Lackawanna and Western Railroad:									
Mikado	5	1.06	1.00	....	1.06	1.14	1.24	1.17	1.20
	15	1.05	0.96	....	1.04	1.18	1.31	1.15	1.14
	25	1.04	1.01	....	1.07	1.16	1.23	1.13	1.13

In general, this average value of the ratio between the stress at the outside edge and the mean stress in base of rail was somewhat greater than unity, although in certain of the wheels it was slightly less than unity. As shown in Table 20, for the Heavy Santa Fe type (Locomotive No. 3813), the ratio under the first driver was 1.14, under the second driver, 1.12, under the third driver (the blind driver), 0.93, under the fourth driver, 1.15, under the fifth driver, 1.05, and under the trailer and tender wheels about 1.05, as averages

for the several speeds. Locomotive No. 3829 gives ratios nearer to unity. It appears that in locomotives of the same class the condition of the tires or other parts has much to do with the amount of the lateral bending of the rail. It is also apparent that different wheels of a locomotive produce lateral bending stresses of quite different magnitudes. At the blind driver, there seems usually to be less outward bending stress than at other drivers. The ratio for the trailer is only a little greater than 1.0.

The first and second drivers of the Light Santa Fe type locomotive present illustrations of very marked outward lateral bending of rail, the ratio of stress at the outside edge of base of rail to mean stress at the first driver being 1.33 at the speed of 5 miles per hour. The ratio for the main driver and the trailer do not vary greatly from 1.00, being 1.01 and 0.96, respectively, at the speed of 5 miles per hour, indicating about as much lateral bending inwardly as outwardly. The average ratio under the first driver is markedly greater for the left rail than for the right. Whether this is due to differences in the condition of the tread of the two drivers on the same axle, to the difference in time of stroke of the two pistons, or to other causes, is not known. There is no marked difference in the amount and nature of the lateral bending of the rails at the two sides of the other locomotives.

As the lateral bending strength of the rail is less than the vertical bending strength, the bending moment corresponding to a given fiber stress is less for lateral bending than for vertical bending. The section modulus, $\frac{I}{c}$, for bending in a vertical plane for the rail sections used is about five times that for bending in a horizontal plane. For a ratio of stress at the outside edge to mean stress with a value of 1.33, it is seen that the lateral bending moment in rail is about 7% of the vertical bending moment required to develop the mean stress in the base of rail. A value of the lateral bending moment found quite frequently is 4% of the vertical bending moment, although in a few cases it reached 14%, if the effect of the added 5 000 lb. per sq. in. of the belt of stresses is taken into consideration. From a study of the observed values, it would seem that in designing a rail section for straight track provision should be made for a lateral bending moment at least equal to 14% of the vertical bending moment.

III.—Tests on Curved Track

12.—The Action of Curved Track.—In traversing a curve, the action of a locomotive or car on the rail and track differs in several respects from the action on straight track. Some of the differences are of importance in the development of stresses in the rail and in the effect on the track structure as a whole. The principal elements of the action of locomotives and cars on curves that need to be considered in the study of the action of curved track may be outlined as follows:

1.—Since the outer rail is longer than the inner one, the wheel on the outer rail must slip backward, or the wheel on the inner rail slip forward, or both, to overcome the difference in travel; this motion of itself may be expected

to be accompanied by a force which acts in a longitudinal direction along the rail rather than laterally, and thus does not produce bending in the rail, although it may modify the magnitude of the force required to produce lateral slipping and hence indirectly affect the magnitude of the lateral bending force.

2.—As the driving wheels of a locomotive, and the wheels of a truck having more than one axle, are grouped in a frame of some stiffness, a lateral movement is involved in changing the direction of the motion of the group of wheels in traversing a curve, and this involves a slipping of one or more pairs of the forward wheels in a lateral direction inwardly of the curve and, generally, of one or more pairs of the rear wheels in a lateral direction outwardly. This action develops lateral pressures against both the inner and the outer rail. A lateral force is thus present at one or more wheels even when the speed is that corresponding to the super-elevation of the track. The lateral pressure thus developed may be of considerable magnitude.

3.—The lateral pressure at two or more of the drivers has been found to give a large increase in the stress at one edge of the base of rail. By reason of the lateral inclination of the track and the centrifugal force developed at the given speed, the division of load between the two rails will not be the same on curved track as on straight track, except at the one speed corresponding to the speed of super-elevation; and, even at that speed, generally the load will not be divided equally between the two wheels of most of the axles. It will appear, also, that there are transfers of load from one rail to the other that are due to other causes.

4.—For some reason, the action of the system of equalizing levers and springs of the locomotive is not the same on curves as on straight track; at least, a much greater vertical bending stress is found in the inner rail under one of the intermediate drivers than under the others, especially at low speeds, and the abnormal action of the equalizers and springs may be one explanation for this. It is also true that a greater vertical bending stress is frequently found in the outer rail under the front driver than under the other drivers, the difference in both cases varying with the speed. This excess of stress under some of the drivers must be due to a large increase of load carried by these drivers.

5.—An effect of the lateral inclination of the track and the presence of centrifugal force is the development of a lateral pressure against the rails; the amount of this lateral pressure may not be equally divided between the two rails or even among the several drivers on one side of the locomotive.

6.—For sharp curves (curves of relatively small radius), there may be a spreading action on the track caused by the flanges bearing against the inner and outer rails; the lower limit of degree of curve above which this spreading action may be expected to become important will depend on the length of wheel base and on the difference between the wheel gauge and the gauge of the track. For the curves in ordinary use, other sources of lateral pressure against the rail appear to be much more important. The several elements of the problem noted will now be considered somewhat more fully.

a.—Difference of Length of Outer and Inner Rail

It is shown by Wellington* that the longitudinal slipping of the wheels of a freight-car truck necessary to overcome the difference in length of the outer and the inner rail of a curve in some cases may occur on the outer rail and in others on the inner rail, although usually not by both wheels of an axle at the same time, as that wheel will slip which will slip more easily, and a slight variation in either the load or the coefficient of friction will give one wheel or the other an advantage in this respect. Wellington further states that either the super-elevation or the centrifugal force is alone competent to produce enough inequality of load to effect a difference and that one wheel having begun to slip is likely to continue so to do for some time, although it may not be the same wheel for two consecutive axles and change from one to the other wheel may take place from time to time. As most of the drivers of a locomotive will be found to be slipping in a lateral direction continually, the matter of the difference between the coefficient of friction for rest and motion will not enter into the question of whether the wheel will have longitudinal slip to the extent that might otherwise be expected from Wellington's analysis for freight-car trucks. The fact that the drivers are coupled together by the side rods is an added feature which may make the action of locomotive drivers differ from that of car trucks. Wellington also shows that the coning of the wheels does not overcome the difference in the length of the rails, even for the curves corresponding to the coning, because the axle usually does not maintain a radial position on the curve, especially when grouped with others in a frame or truck; the coning, therefore, may be neglected as an element in the problem. The slipping of the wheel longitudinally on the rail results in a force being developed in the direction of the rail length (pull or push) which adds to the tractive pull required and produces wear of rail and wheel. It does not, however, cause bending stresses in the rail and, therefore, need not here be taken into account, except as it may affect the way in which lateral slip on the rail occurs in combination with it and also as it may influence the magnitude of the lateral force, which is one component of the force required to produce the resulting slip on the rail.

b.—Changing the Direction of the Locomotive in Passing Around the Curve

If in traversing a curve the axles of each pair of wheels took a radial direction (normal to the curve), the principal force required to change the direction of the motion of the truck or frame would be equal to that commonly called the centrifugal force; the centrifugal force and the transverse inclination of the track cause a lateral reaction against the rail. For a radial position of the axle, the force producing lateral bending in the rail would then be the resultant of the centrifugal force and the component of the load due to the transverse inclination of the track.

When several axles are connected to a single stiff frame, as in a locomotive, quite different conditions are set up. The axles of the drivers must remain

* "The Economic Theory of Railway Location", p. 285.

parallel to each other. The tendency of the whole group of drivers to go straight ahead must be overcome by a lateral force exerted by the outer rail against the flanges of one or more of the wheels at the front of the frame. In the case of the locomotive, this turning movement is effected at the outer front driver, or at the outer wheels of the front truck, or by a combination of these truck wheels and the first driver. What wheel or combination of wheels will participate in this turning action may be expected to depend on the curvature of the track, the design of the locomotive, and the stiffness and play of the front truck connection with the locomotive frame, and, also, in some cases, on the flexibility of the connections of the front pair of drivers with the other drivers of the locomotive. The action of these forward outer wheels therefore is to press against the outer rail, the outward lateral force thus exerted tending to increase the curvature of the outer rail; in some cases, by reason of friction on the rail, the truck wheel on the inner rail may exert a lateral pull in the same direction on that rail. As a result of this turning action, the front drivers (both inner and outer) will be made to slip laterally on the rail; that is, inwardly radially to the curve. Farther back along the frame of the locomotive will be a point which will act as a center of rotation and about which the turning of the locomotive will take place. If the lateral slip at any wheel is inward with reference to the curve (that is, toward the center of the curve), the effect on the rail is that of a lateral force acting inwardly of the curve, thus putting an inward lateral pressure on the inner rail and even on the outer rail, since the force may be transmitted by a wheel to the rail by friction at the top of the rail. The foregoing statement does not apply, of course, to those outer wheels the flanges of which are active in causing the locomotive to change direction; for these the pressure against the rail is outward, tending to increase the curvature of the rail.

The lateral slip of the drivers in front of the point about which rotation takes place will be inward with reference to the curve; that of the drivers back of the point will be outward. It is apparent that the position of this center of rotation will depend on the number of wheels in the frame, the position the wheels take with respect to the two rails, and the direction of the resultant slip of the wheels. If, as may happen, the axle of the next to the last pair of drivers remains normal to the curve, the center of rotation will be on the inner rail at this driver, provided, as is probable, that all the longitudinal slip of the drivers of this axle is taken by the outer driver. It seems probable that the center of rotation in some cases may be somewhat ahead of the next to the last driver. It is true, of course, that the several drivers on one side may not lie exactly in a straight line; the lateral play of the journals will allow side movement, the amount depending on the amount of the wear at the time. Fig. 38 represents the position of the wheels and the direction of a part of the lateral forces exerted by the rails on the wheels of the Mikado type locomotive at low speeds under conditions similar to those named. It is seen that the point of rotation is at or close to the third driver axle. The greatest lateral slip for any driver may be expected to occur under the front drivers. The

lateral slip of the several drivers will then be proportional to the distance along the inner rail from the point of rotation to the inner driver of a given pair.

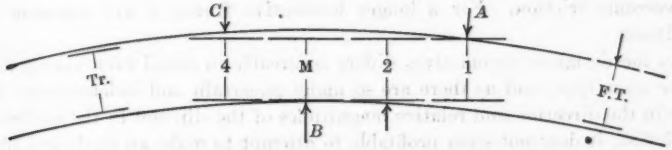


FIG. 38.—POSITION OF THE WHEELS OF A MIKADO TYPE LOCOMOTIVE WHEN TRAVERSING A CURVE AT LOW SPEEDS.

Generally, the two-wheel front truck is connected to a radius bar, a swinging arm that is pivoted at a point back of the truck, and a limit of rotation is provided, so that after this limit is reached the flange of the outer wheels will press against the outer rail and thus aid in the turning action. To what extent the axles of the front truck will approximate a radial condition on the curve and thus will have minimum lateral slip will depend on the manner of the attachment and the sharpness of the curve. The trailing wheels are also pivoted on a swinging arm so that the axle possibly may take a radial position. When the limit of the swing of this arm is reached, these wheels also contribute to the stiffness of the frame and must be slid outwardly; thus an outward lateral force on the rails is developed at the trailers.

In Figs. 73 and 74 of Article 15, "Lateral Bending of Rail on Curves", the position taken by the drivers and other wheels on curved track is shown for several locomotives. A study of these diagrams and of the amount and direction of the bending of the rails will indicate the position of the center of rotation of the locomotive. The relative amounts of the lateral slip at the several wheels may also be found rather closely.

It should be noted further that as both longitudinal and lateral slip on the rail will usually occur at the same time, resulting in slip in a diagonal direction, the force producing this slip of wheel in the diagonal direction (its magnitude being the force necessary to overcome friction) may be resolved into components along the rail and across the rail. In Fig. 39, the diagonal

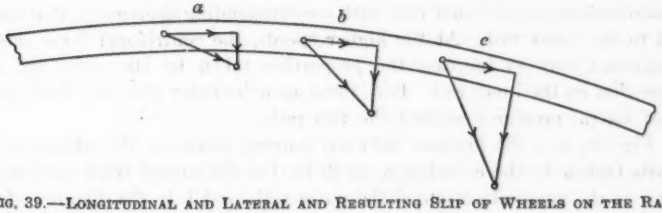


FIG. 39.—LONGITUDINAL AND LATERAL AND RESULTING SLIP OF WHEELS ON THE RAIL.

line represents relatively the direction and amount of the resultant slip, the line along the rail the longitudinal slip, and the line drawn transversely of the rail the lateral slip. It is apparent that the magnitude of the lateral component of the force producing slipping is dependent on the relative magnitudes of the longitudinal and lateral slip. For the conditions at *a*, the lateral com-

ponent or lateral pressure against the rail will be small. For equal slip in the two directions, as at *b*, which may be the condition for a short truck like a four-wheel car truck, the lateral component will be 0.7 of the force required to overcome friction. For a longer locomotive frame, *c* will represent the conditions.

As the design of locomotives differs so greatly in detail even among those of the same type, and as there are so many uncertain and indeterminate features in the direction and relative magnitudes of the slip and in the coefficients of friction, it does not seem profitable to attempt to make an analytical treatment of the lateral forces developed at the several wheels by the turning of the locomotive, although it would be easy to show that the magnitude of the lateral forces developed is necessarily considerable. Instead, it seems better to consider the data of the stresses observed in the rails themselves and to discuss these data in the light of the action herein outlined. It may be added that analytical considerations indicate that, for the same super-elevation and at low speeds, the degree of curve has little effect on the nature and magnitude of the turning forces beyond a lower limit of curvature where the assumed condition comes into action and within an upper limit where other elements may exert a controlling influence.

It should be pointed out that these turning forces, together with the centrifugal force and the component due to the transverse inclination of the track and the forces required to produce lateral slip, and including the re-acting force at the inner rail, are the principal and most important lateral forces producing bending moment and bending stress in the two rails of curved track.

c.—Division of Load by Reason of Transverse Inclination of Track and Centrifugal Force

On straight track having the rails at the same level transversely, it may be presumed that the load on one axle is transmitted in equal amounts to the two rails. On curves an equal division between the two rails may not be generally expected. At low speeds (the centrifugal force being negligible), the super-elevation of the outer rail results in more than one-half the load being transmitted to the inner rail, with a corresponding decrease in that transmitted to the outer rail. At the higher speeds, the centrifugal force acts in an important way to increase the proportion taken by the outer rail and decrease that on the inner rail. Both these agencies enter also into the development of lateral pressures against the two rails.

In Fig. 40, g is the distance between bearing points of the wheels on the two rails (taken in the calculation as 59 in. for the curved track used in the tests); e is the super-elevation of the outer rail; and h is the distance of the center of gravity of the locomotive from the level of the top of the rails. Use W as the weight which would be applied to the rail through one wheel for straight and level track, and R' the reaction on the outer rail and R'' that on the inner rail which would be expected from analytical considerations to correspond to the weight, W. Then, if the speed is such that the centrifugal force

is negligible, it may be shown that the load transmitted to the outer rail will be given approximately by the following equation, the cosine of the angle of the transverse inclination of the track being taken as unity,

$$R' = W\left(1 - 2\,\frac{h}{g}\,\frac{e}{g}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)^*$$

and that transmitted to the inner rail,

$$R'' = W\left(1 + 2\,\frac{h}{g}\,\frac{e}{g}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

For $h = 72$ in. and $e = 6$ in., Equations (29) and (30) indicate that for low speeds the load transmitted to the outer rail is 25% less than that on track level transversely and that to the inner rail is 25% greater than the normal load. It is evident that the inequalities in the division of load between the two rails may be a matter of some consequence.

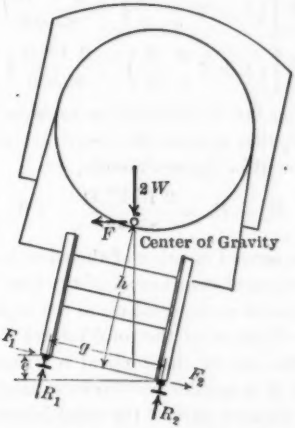


FIG. 40.—FORCES AND REACTIONS ON CURVED TRACK HAVING TRANSVERSE INCLINATION.

Equations (29) and (30) were derived on the assumption that the body of the locomotive retains the same position with respect to the axles of the drivers that it has on straight track. If, as may be the case, the greater weight on the inside of the curve causes the springs on that side of the locomotive to deflect more, and those on the outside of the curve less, than would normally be found, the result will be to tilt the locomotive still further and thus to throw greater weight on the inner rail than is given by Equations (29) and (30). The effect of this tilting on the transfer of load is less accordingly than that due to transverse inclination of track alone, as the tilting action is about the point of attachment of the equalizing levers to the frame of the locomotive, which is somewhat above the level of the axles of the drivers. As the springs deflect less proportionately for an overload than for a small load, the effect of this

* This equation number follows the last number used in the second progress report, *Transactions*, Vol. LXXXIII (1919–20), p. 1574.

change in deflection of the springs will not be as great as would otherwise be expected. It, however, may need some consideration.

For the higher speeds, the centrifugal force tends to counteract this inequality. For the speed corresponding to the super-elevation given to the track, the inequality vanishes, and it would be expected that the rails will receive equal loads. For still greater speeds, the outer rail will receive the greater load. In any case, the load on a rail would be expected to be the algebraic sum of that due to Equation (29) or Equation (30) and that due to the centrifugal force. The centrifugal force at V miles an hour on a $D°$ curve due to the weight, $2W$, is, approximately, $F = \frac{2WV^2D}{85\,800}$. Consider this force to be applied at the center of gravity of the locomotive. The reaction of the outer and inner rails, respectively, will then be:

$$R_1 = W\left[\left(1 - 2\frac{e}{g}\frac{h}{g}\right) + \frac{2V^2D}{85\,800}\left(\frac{h}{g} + \frac{1}{2}\frac{e}{g}\right)\right] \quad \ldots\ldots (31)$$

$$R_2 = W\left[\left(1 + 2\frac{e}{g}\frac{h}{g}\right) - \frac{2V^2D}{85\,800}\left(\frac{h}{g} - \frac{1}{2}\frac{e}{g}\right)\right] \quad \ldots\ldots (32)$$

If F_1 is the resulting lateral reaction or pressure against the outer rail of the curved track and F_2 that against the inner rail, the sum of the lateral reactions for the two rails will be approximately,

$$F_1 + F_2 = \frac{2WV^2D}{85\,800} - 2W\frac{e}{g} \quad \ldots\ldots (33)$$

The first term of the second member of the equations is the centrifugal force and the second the horizontal component of the load due to super-elevation of the outer rail. At the speed corresponding to the super-elevation, $F_1 + F_2 = 0$. It is apparent that the division of the total lateral force between F_1 and F_2 is indeterminate, as in the case of the vertical reactions of the two hinges of a door. Generally, a low or a smaller resistance to lateral slip on one wheel may be expected to cause a greater part of the total lateral force to go to the other rail, except as modified by the bearing of the flange of the wheel against the side of the rail. It will be seen from Equation (33) that the value of $F_1 + F_2$ for the highest super-elevation of the track used in the tests, 8½ in., is 14% of the total load on the axle.

Values of the ratio of the vertical load on the outer and inner rail of curved track to the corresponding value on straight track, as found by Equations (31) and (32), are given in Tables 21 and 22 for the curves and speeds used in the tests.

d.—Spreading Action of Drivers and Other Wheels

The stiffness of the frame and the smallness of the lateral play in the journals when the locomotive is just out of the shops may leave little individual freedom of lateral movement in the drivers. With a long and rigid wheel base, it may be thought that on sharp curves a flange of an intermediate driver will be crowded against the inner rail while the flanges of the end drivers are pressed against the outer rail, and, thus, that there would be a marked tendency

to spread the rails apart, this action creating lateral bending forces against the two rails. As the gauge of the wheels is less than that of the track (the difference ranging from ¾ in. to 1 in. for new wheels and sometimes as much as 2 in. for badly worn flanges and widened gauge), and as on curves there may be an increase in gauge over the standard, and as there may be some flexibility in the frame and bearings, it is to be expected that, for a given wheel base and other stated conditions, the limiting degree of curve below which no spreading action exists of the kind here meant, will be rather large. For the purpose of giving greater freedom, the flange is sometimes omitted from one or more drivers, making so-called blind drivers.*

TABLE 21.—Ratio of Loads on Inner and Outer Rail, Atchison, Topeka and Santa Fe Railway.

Locomotive.	Degree of curve.	Super-elevation, in inches.	Speed, in miles per hour.	Ratios. Analytical Values. Inner rail.	Analytical Values. Outer rail.	Test Values. Inner rail.	Test Values. Outer rail.
Ribera:							
Mountain, No. 3710.....	6°	4.7	5	1.20	0.80	1.22	0.78
			25	1.09	0.91	1.16	0.84
			35	1.00	1.00	1.02	0.98
			50	0.77	1.26	0.81	1.19
Santa Fe, No. 3813.....	6°	4.7	5	1.29	0.80	1.19	0.81
			25	1.09	0.91	1.11	0.89
			40	0.92	1.09	1.01	0.99
Mountain, No. 3710.....	10°	4.7	5	1.19	0.81	1.21	0.79
			25	1.02	0.99	0.99	1.01
			35	0.85	1.17	0.81	1.19
Santa Fe, No. 3813.....	10°	4.7	5	1.19	0.81	1.20	0.80
			25	1.02	0.99	1.00	1.00
			35	0.85	1.17	0.83	1.17
Santa Fe, No. 3829.....	10°	4.7	5	1.19	0.81	1.28	0.72
			25	1.02	0.99	1.00	1.00
			35	0.85	1.17	0.88	1.12
						Up Grade. / Down Grade.	Up Grade. / Down Grade.
Bealville:							
Santa Fe, No. 3832.....	10°	4.4	10	1.16	0.84	1.15 / 1.15	0.85 / 0.85
			18	1.10	0.91	1.09 / 1.08	0.91 / 0.92
Santa Fe, No. 3653.....	10°	4.4	10	1.16	0.84	1.18 / 1.18	0.82 / 0.82
			18	1.10	0.91	1.13 / 1.12	0.87 / 0.88
Cajon:							
Santa Fe, No. 3832, fourth driver cylindrical.....	10°	5.4	10	1.20	0.80	1.27 / 1.36	0.73 / 0.64
			18	1.14	0.86	1.17 / 1.30	0.83 / 0.70
Santa Fe, No. 3832, fourth driver grooved.....	10°	5.4	10	1.20	0.80	1.32 / 1.32	0.68 / 0.68
			18	1.14	0.86	1.18 / 1.24	0.82 / 0.76

To find whether a given curve is below the limiting value for a given wheel base on the basis that the flanges bear against the two rails in the manner outlined, the ordinates for the curve may be found. In Fig. 41, the flanges of the outer end drivers are both considered to bear against the outer rail at

* For a discussion bearing on the widening of gauge on curves, see Report of Committee V on Track, *Proceedings*, Am. Ry. Eng. Assoc., Vol. 23, p. 575.

points which may be somewhat beyond the limits of the wheel base, and the flanges of intermediate wheels against the inner rail. The ordinates, a and b, are then found. Then, if g is the gauge of the track and g' the gauge of the wheels, the distance, a-b, must be enough greater than g-g' to give reasonable clearance. For five drivers on a side, the term, b, vanishes if the middle driver is flanged; with a blind driver at the middle, the chord from the second to the fourth driver will be used in obtaining a value of b.

TABLE 22.—Ratio of Loads on Inner and Outer Rail, Delaware, Lackawanna and Western Railroad.

Locomotive.	Degree of curve.	Super-elevation, in inches.	Speed, in miles per hour.	Ratios			
				Analytical Values.		Test Values.	
				Inner rail.	Outer rail.	Inner rail.	Outer rail.
Mikado, No. 1220	4°	3.7	5	1.16	0.84	1.03	0.97
			25	1.08	0.92	1.01	0.99
			35	1.01	0.99	0.97	1.03
			45	0.92	1.08	0.89	1.11
Pacific, No. 1131.	4°	3.7	5	1.16	0.84	1.11	0.89
			25	1.08	0.92	1.04	0.96
			40	0.97	1.03	0.94	1.06
			60	0.74	1.28	1.00	1.00
Mikado, No. 1220	6°	8.5	5	1.36	0.64	1.37	0.63
			25	1.26	0.75	1.30	0.70
			35	1.16	0.86	1.20	0.80
			45	1.02	1.01	1.16	0.84
Mikado, No. 1220	7½°	6.4	5	1.27	0.73	1.29	0.71
			25	1.14	0.87	1.19	0.81
			35	1.01	1.00	1.10	0.90
			45	0.84	1.19	0.97	1.03
Pacific, No. 1131.	7½°	6.4	5	1.27	0.73	1.26	0.74
			25	1.14	0.87	1.16	0.84
			40	0.93	1.09	0.96	1.04
			50	0.74	1.30	0.87	1.13

The assumption that the outer rear driver bears against the outer rail is not warranted by the observations and tests made on curved track; the rear driver generally keeps well away from the outer rail at low speeds and usually also at the higher speeds. This position of the rear driver in effect lengthens the wheel base and decreases the degree of curve for which clearance would be found sufficient if other elements did not enter into the phenomenon. An important modification of the assumptions is that the lateral deflection of the rails at some of the wheels is found to be considerable, particularly in the

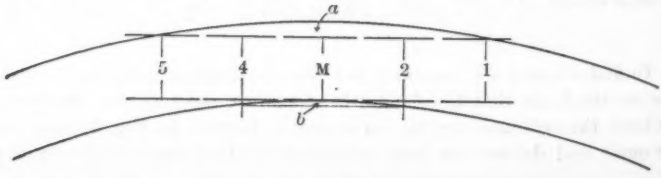


Fig. 41.—Mid-ordinates of Portion of Curve Between Certain Drivers of a Locomotive.

inner rail at a rear intermediate driver. With the Mountain type locomotive, on a 10° curve, the inner rail was found to deflect inwardly of the curve at the third driver as much as 0.5 in., the gauge of the track increasing to more than that extent because the outer rail also deflects somewhat. The Santa Fe type locomotive with all drivers flanged and the Santa Fe type with the main driver flangeless appeared to traverse the 10° curves without the flanges of the intermediate drivers bearing against the inner rail, and hence there was no spreading action of the nature of that usually considered when this subject is discussed. As a very heavy thrust against the inner rail on 10° and even on 6° curves is found—really an excessive pressure—it is plain that an explanation of its source other than the spreading action of the flanges must be sought.

The explanation that appears plausible is connected with the change of direction of the locomotive. Consider in Fig. 38 that the force changing the direction of the locomotive is applied by the rail to the flange of the outer first driver at *A*, or by the wheels of the front truck at some point ahead of this. Consider that the rotation is about the point, *B*, on the inner rail and that the rear outer driver is slipped outwardly at *C*. The lateral pressure at *B* will have to resist the forces at *A* and *C*, except as other wheels may assist. As the other wheels slip laterally, if the wheel at *B* does not slip, as seems to be the case, the main part of the resisting lateral thrust will be taken at *B*. This seems to be the more plausible, when, as is found in most tests at low speed, the load transmitted by the driver in question is greatly in excess of that taken by the others. It will be found also from some of the tests with the Santa Fe type locomotive that the conditions of this lateral thrust change materially when all ten drivers are flanged and when four of them are flangeless. Measurements of the lateral deflection of the inner and outer rails of curved track confirm the action of the lateral forces referred to previously and show a considerable increase in gauge without the flanges touching the inner rail.

Altogether, it seems to be evident that the limiting conditions of curves cannot be determined by a study of ordinates and clearances alone. The absence of wear on the gauge side of the inner rail of such curves of itself would seem to absolve the flanges of the drivers from spreading action on the inner rail. The presence of very great lateral pressures against the inner rail is quite evident in the tests. These pressures produce a widening of the gauge, something like a spreading action, but the origin of the pressure should be attributed to sources other than lack of clearance of flanges of the intermediate drivers. It seems clear that the lateral force is transmitted by the drivers to the inner rail by means of the friction between the tread of the driver and the head of the rail. The very high vertical pressures transmitted by one or more of the inner drivers and the greater value of static friction combine, in many cases, to make the point of contact of the rear intermediate driver on the inner rail the center of rotation of the change of direction of the locomotive. Thus, this driver is kept from slipping laterally while, at the same time, the great lateral pressure on the inner rail developed at this driver causes the lateral deflection and spreading of the inner rail to be considerable, even on

track of moderate curvature. In general, it is evident that the method of calculating clearances indicated in Fig. 41 is not generally applicable.

e.—Other Sources of Lateral Pressure

When the locomotive is pulling a load around a curve, the tractive pull of the locomotive at the coupling with the tender will not be in the direction of the tangent to the curve at that point, in other words, there will be obliquity of traction. This obliquity of traction will produce a lateral force on the rear end of the locomotive itself, inwardly of the curve, a force that is the component of the tractive force in the direction of the radius of the curve. The approximate direction of the frame of the locomotive may be found by considering that the frame is tangent to the curve at the point of rotation of the turning movement of the locomotive around the curve, that has already been referred to. The point of rotation has been found to be at or near the next to the last inner driver. For the Santa Fe type locomotive, on a 10° curve, the angle of the direction of the locomotive with the tangent to the curve would then be not more than 2°, and probably less, and for the other locomotives and for lighter curves, the angle would be smaller still. It will be seen that under these conditions the lateral component of the tractive force may be expected to be small in comparison with the other loads and forces acting. The effect of this lateral component applied at the point of the coupling will be to tend to swing the rear end of the locomotive inwardly of the curve and the front end outwardly. The effect at the rear drivers will be to relieve the outward lateral thrust on the rail at that point if these drivers do not slip outwardly, and to increase the lateral force acting on the locomotive at that point if the rear drivers do slip outwardly. Both conditions would tend to increase the lateral inward thrust of the driver at the point of rotation and also the outward lateral thrust at the front driver or front truck wheel. From the small value of the angle of obliquity and the possible magnitude of the tractive force, it seems very probable that all the lateral thrusts due to obliquity of traction are small. Whether they actually have any serious bearing on the lateral bending of the rail may best be found from experiments. It may be added that Wellington in the discussion of the effect of obliquity of traction on train resistance came to the conclusion that the additional force required to pull the train due to this cause was too small to need consideration.

On straight track, there are ordinarily lateral movements of locomotives and cars, or so-called vibrations, which cause lateral bending stresses in the rail. These may be due to variations in the track, or to lateral motion of the locomotives and cars. The lateral bending stresses produced in this way may be of considerable magnitude, as has already been shown. On curved track, the lateral outward and inward movements may be expected to be less than those on straight track, because of the action of the lateral component of the load due to the transverse inclination of the track and of the lateral forces developed in the turning of the locomotive. Even at the speed of super-elevation the lateral forces will be quite effective in keeping the wheels in a given position. It is true, of course, that if the curved track is not in well-kept con-

dition of line and super-elevation, the lateral movement may be much greater than on straight track, as when the locomotive and cars careen from side to side.

It should be understood that the discussion of the action of curved track here made is preliminary to the discussion of the results of the tests; no analytical treatment can have finality.

13.—General Results of Tests on Curved Track.—In so complicated a problem, it is not easy to present the data of the tests, interpret the meaning of the results, and distinguish between the effects of the various factors which together influence the action of the locomotive and track. It is to be expected then that there may be some repetition or duplication of statement in the discussion. Space will permit the presentation of only a part of the data, but representative parts have been selected and general average values are given in the tables and diagrams.

Figs. 42 to 63, inclusive, give the stresses at the inside edge and the outside edge of the base of the inner and the outer rail under the wheels of six locomotives at several speeds throughout the revolution of the driver in the tests on the Atchison, Topeka and Santa Fe Railway and the Delaware, Lackawanna and Western Railroad. The observations from which these averages were obtained, when plotted with respect to the revolution of the driver, were found to be somewhat more scattered than is the case with straight track, as is to be expected from the greater number of variable factors entering into the problem, but the body of the observed values lies in a belt that is frequently within 6 000 lb. per sq. in. of stress of the line of average stress although, in some cases, the range is much wider. This is true for diagrams representing mean stress, stress at inner edge, and stress at outer edge of base of rail. It is evident then that, as in the case of straight track, the line or curve of average stress may be taken to be representative of the values, with the understanding that values may frequently be expected greater and less than this line of average stress within the limits of the belt, with occasional values greater or less than these limits, and that the range of stress above and below the average line may be taken to be 4 000 to 12 000 lb. per sq. in. for the rail sections and loads used in the tests.

In Figs. 64 to 67, inclusive, are given the stresses at the inside edge and the outside edge of the base of the inner and the outer rail under the several wheels of three types of locomotives at the speeds used in the tests.

In making comparisons between tests, reference may be made to the properties of the rail sections given in Table 1 and the weights of the locomotives given in Figs. 1 to 3.

From an examination of the diagrams, it is evident that there are marked differences in stresses under the several drivers, that these differences vary greatly with speed, and that there are high lateral bending stresses in the rail under certain wheels.

14.—Stresses Due to Vertical Bending of the Rail.—The mean stress in the base of rail (the average of the stresses observed at the two edges and generally hereafter called the vertical bending stress), may be taken as represen-

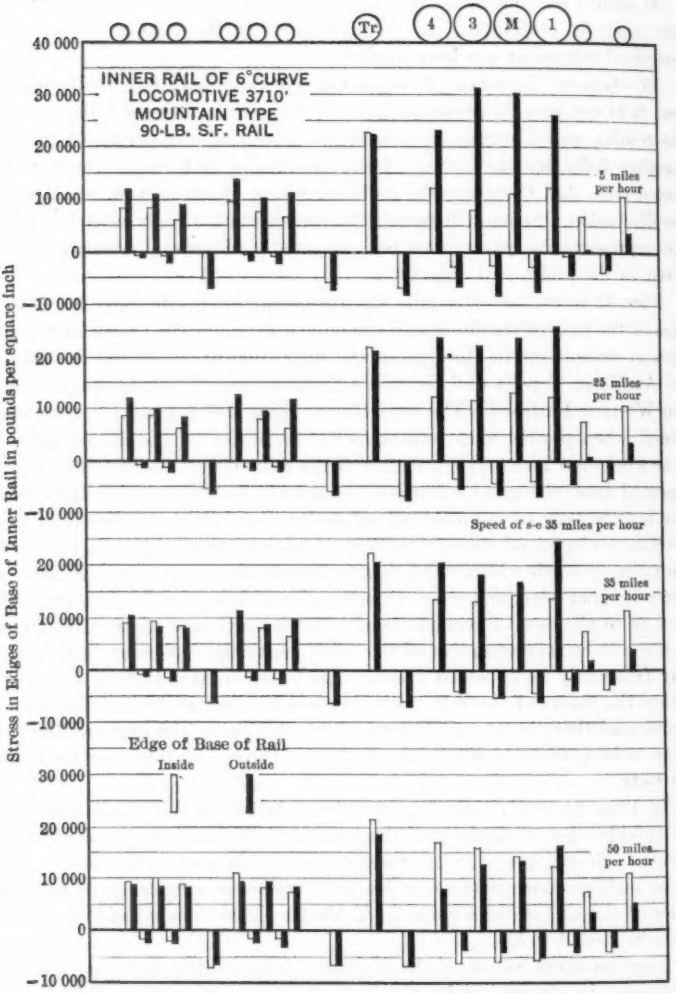


FIG. 42.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER RAIL OF THE 6° CURVE, SERIES 5319–5337, MOUNTAIN TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

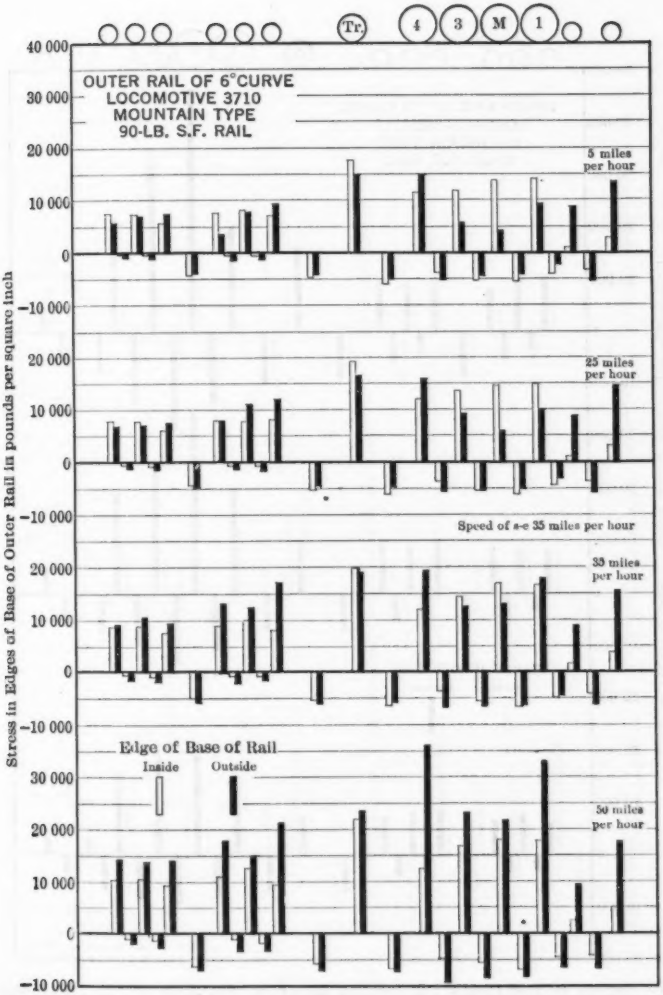


FIG. 43.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE OUTER RAIL OF THE 6° CURVE, SERIES 5319–5337, MOUNTAIN TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

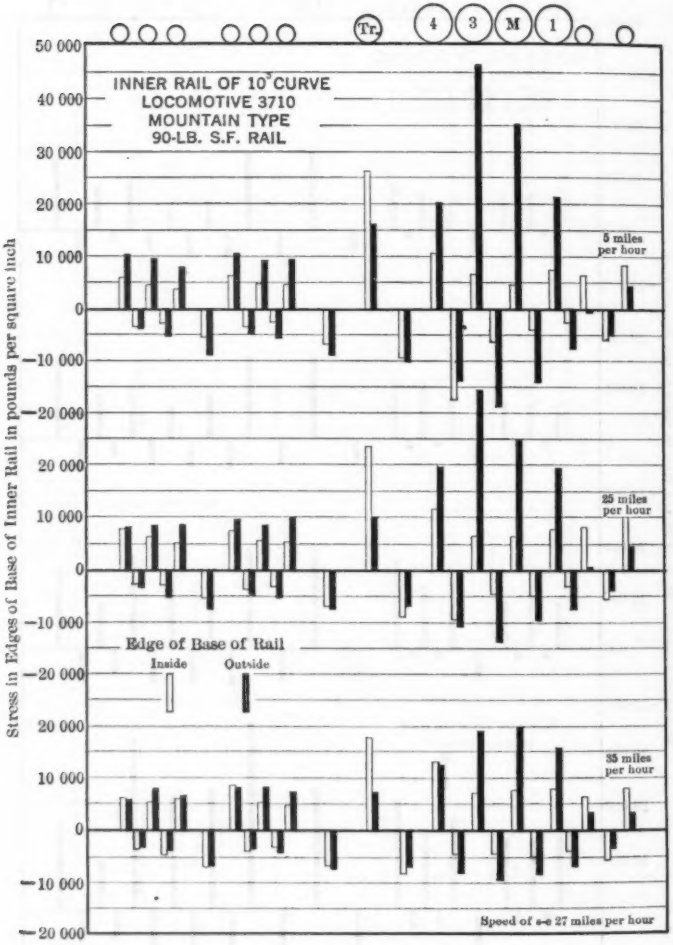


FIG. 44.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER RAIL OF THE 10° CURVE, SERIES 5351–5365, MOUNTAIN TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

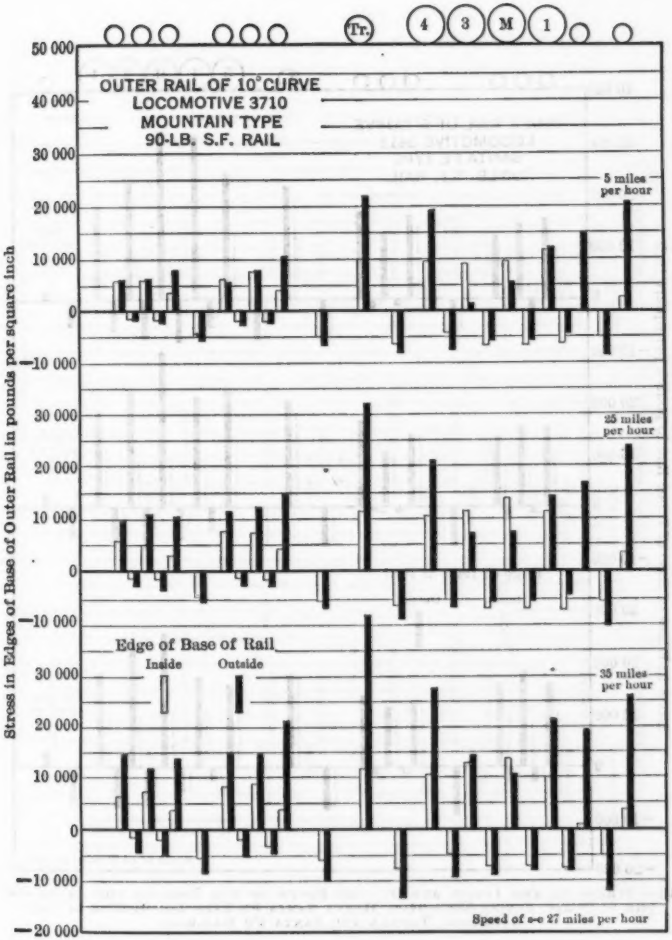


FIG. 45.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE OUTER RAIL OF THE 10° CURVE, SERIES 5351–5365, MOUNTAIN TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

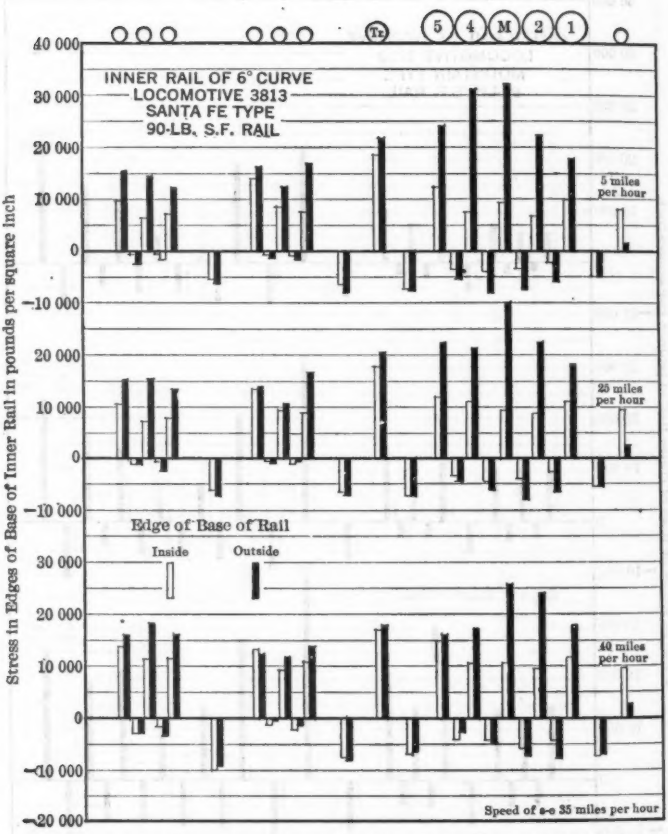


FIG. 46.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER RAIL OF THE 6° CURVE, SERIES 5383–5400 HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

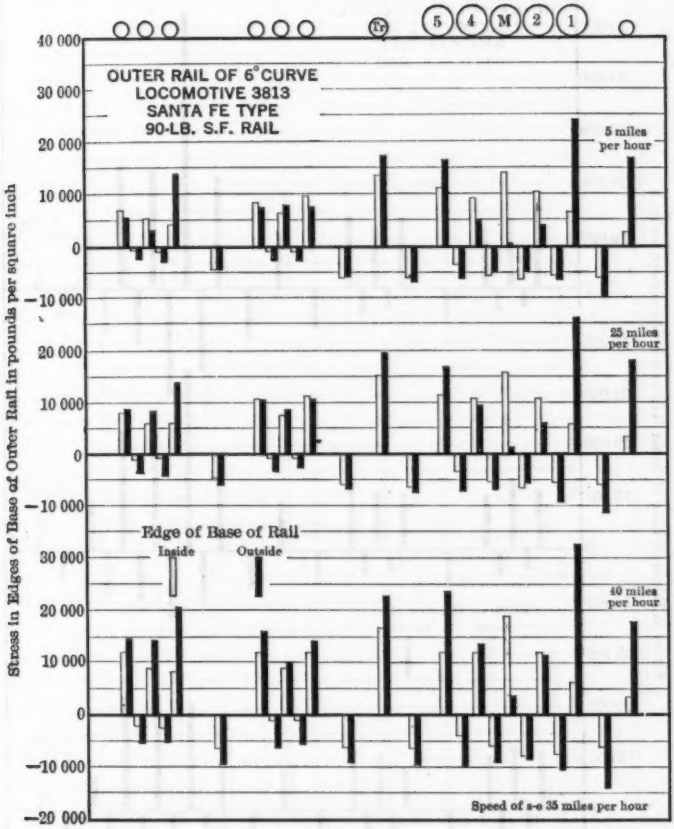


FIG. 47.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE OUTER RAIL OF THE 6° CURVE, SERIES 5383–5400, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

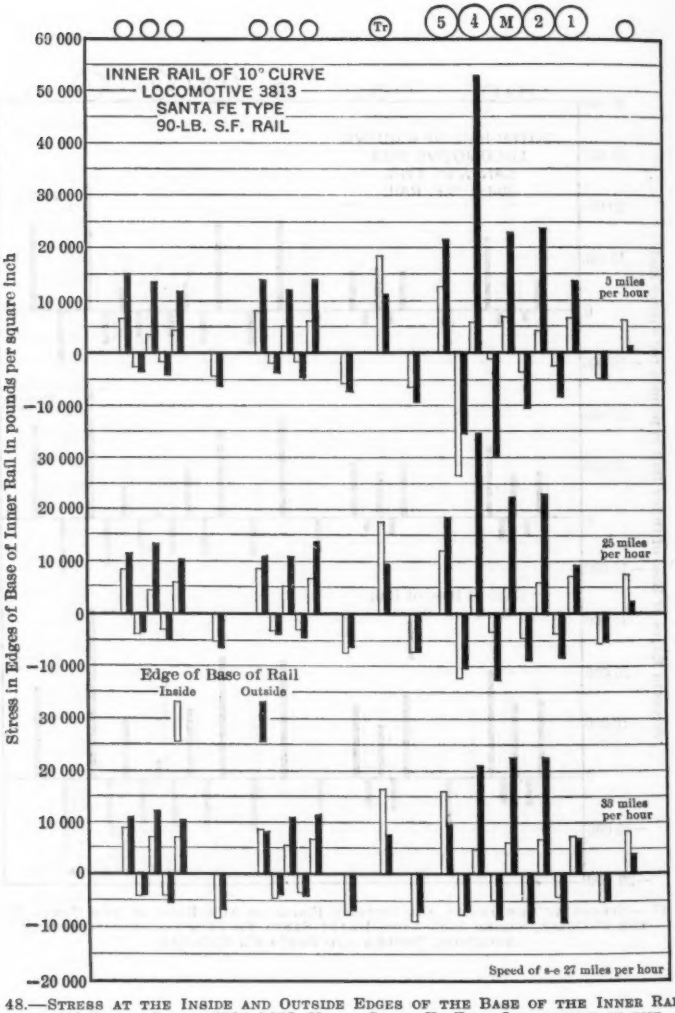


FIG. 48.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER RAIL OF THE 10° CURVE, SERIES 5366–5382, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

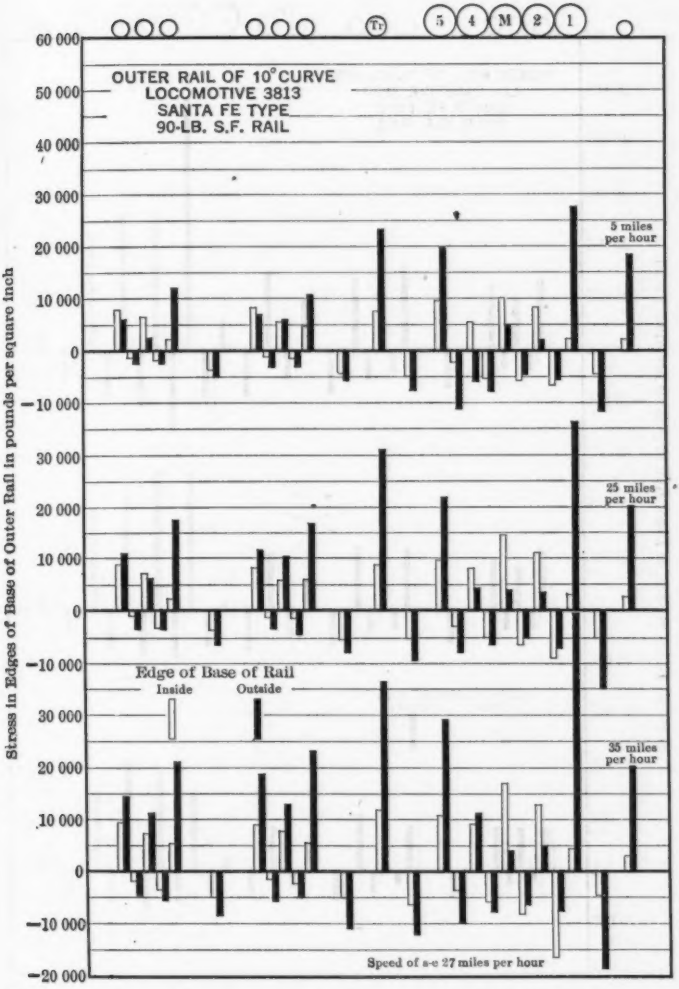


FIG. 49.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE OUTER RAIL OF THE 10° CURVE, SERIES 5366–5382, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

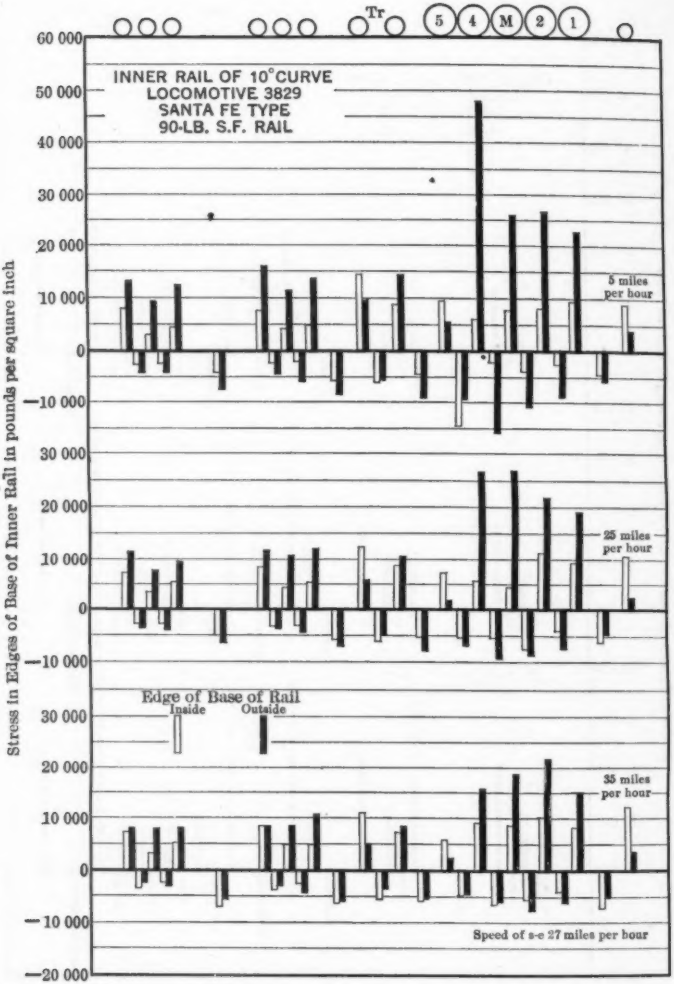


FIG. 50.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER RAIL OF THE 10° CURVE, SERIES 5425–5433, DOUBLE TRAILER HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

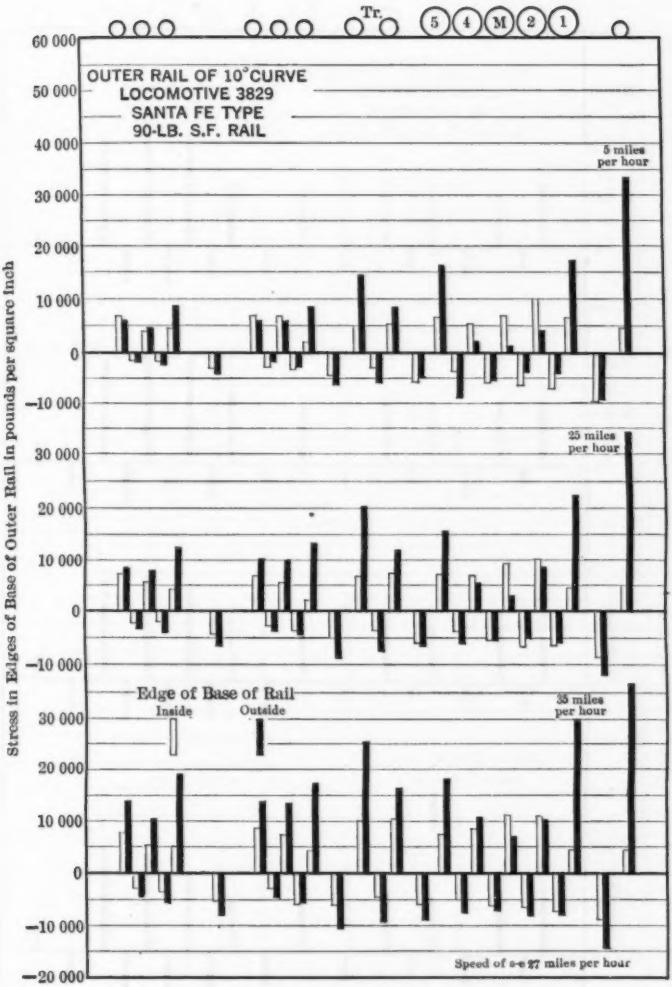


FIG. 51.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE OUTER RAIL OF THE 10° CURVE, SERIES 5425–5433, DOUBLE TRAILER HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

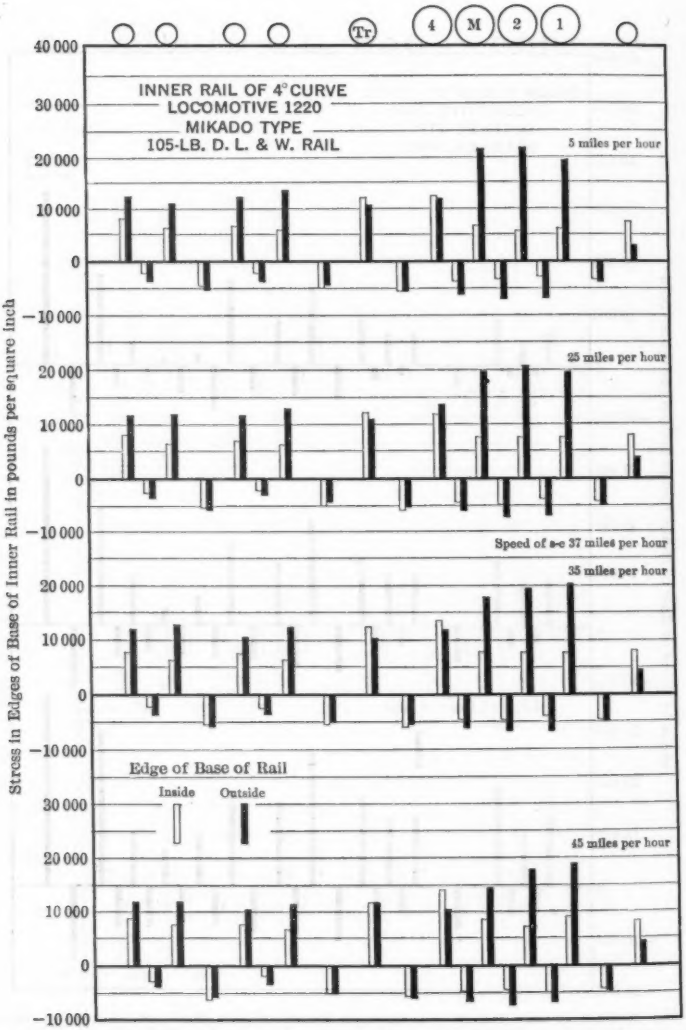


FIG. 52.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER RAIL OF THE 4° CURVE, SERIES 5150–5169, MIKADO TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

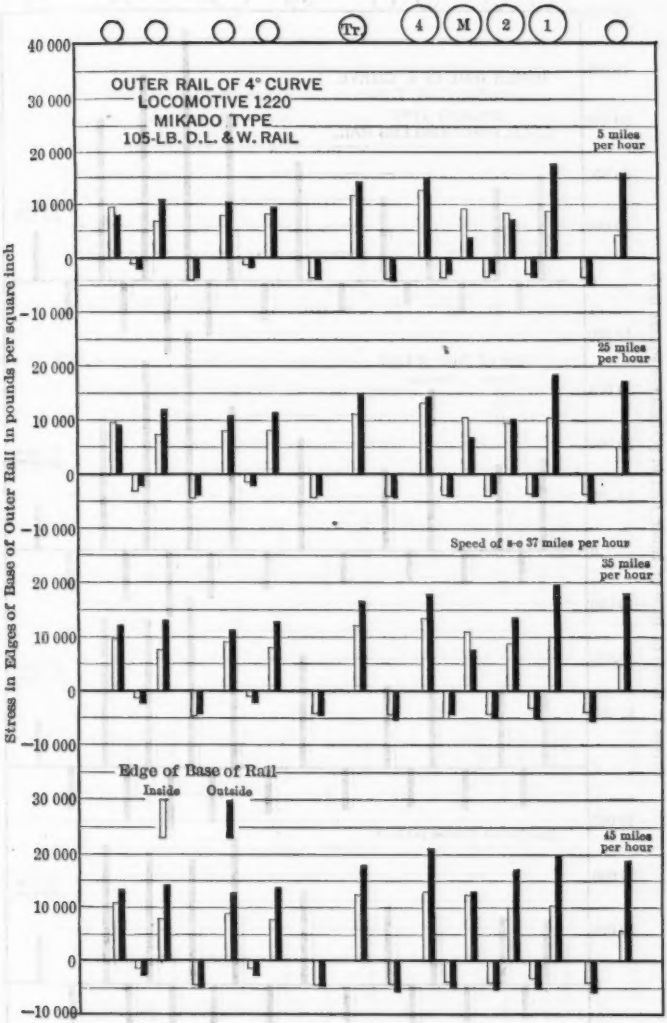


FIG. 53.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE OUTER RAIL OF THE 4° CURVE, SERIES 5150–5169, MIKADO TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

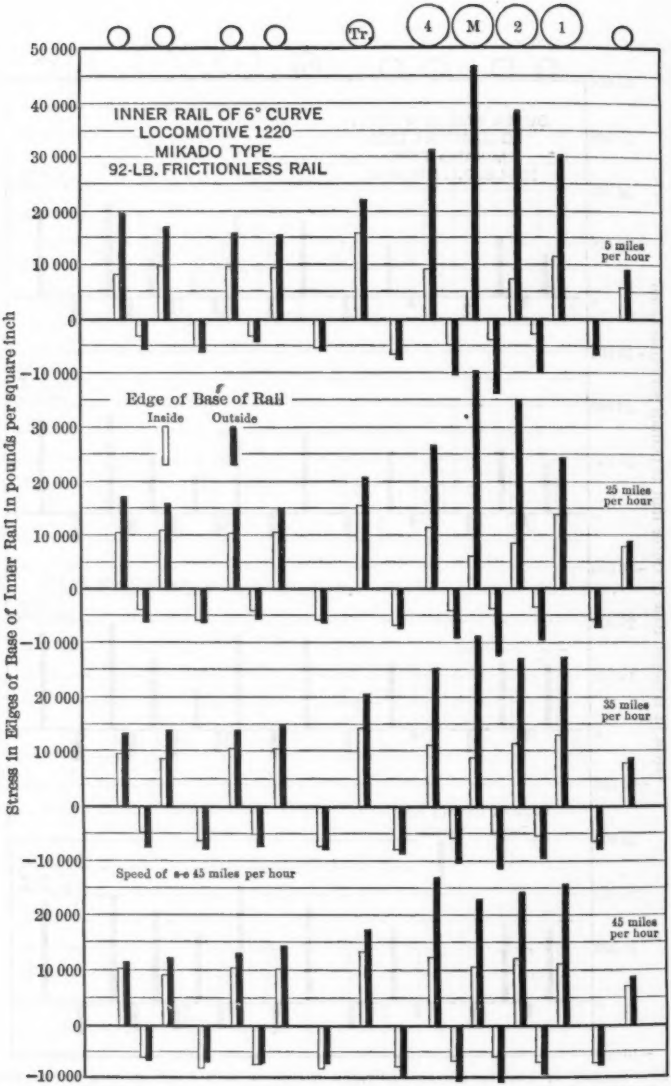


FIG. 54.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER RAIL OF THE 6° CURVE, SERIES 5191–5205, MIKADO TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

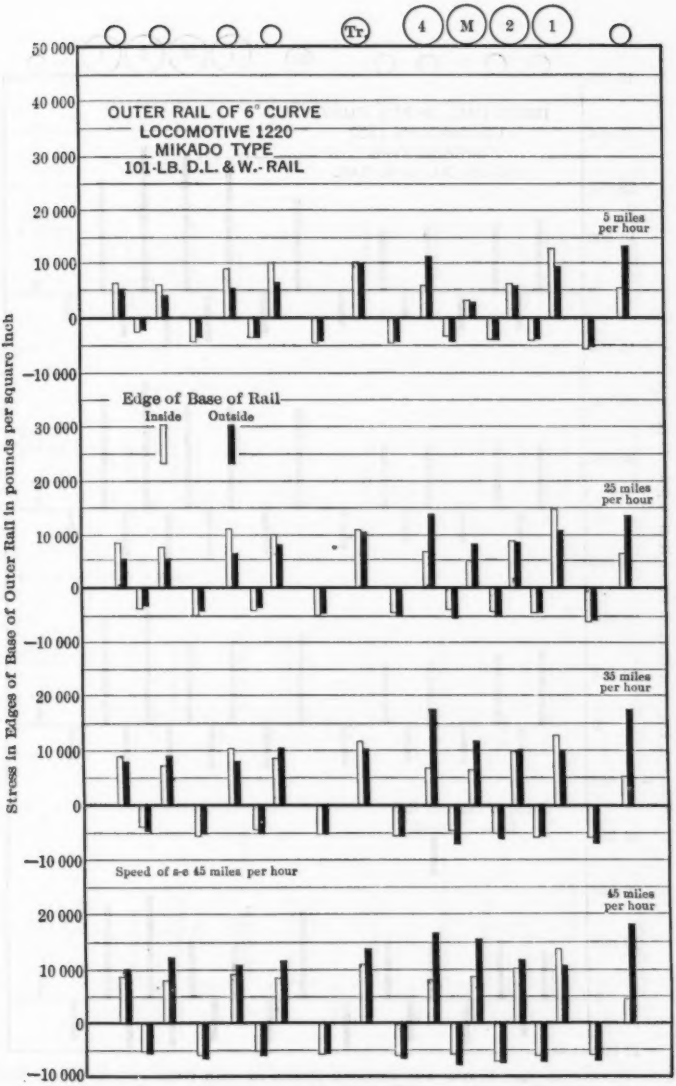


FIG. 55.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE OUTER RAIL OF THE 6° CURVE, SERIES 5191–5205, MIKADO TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

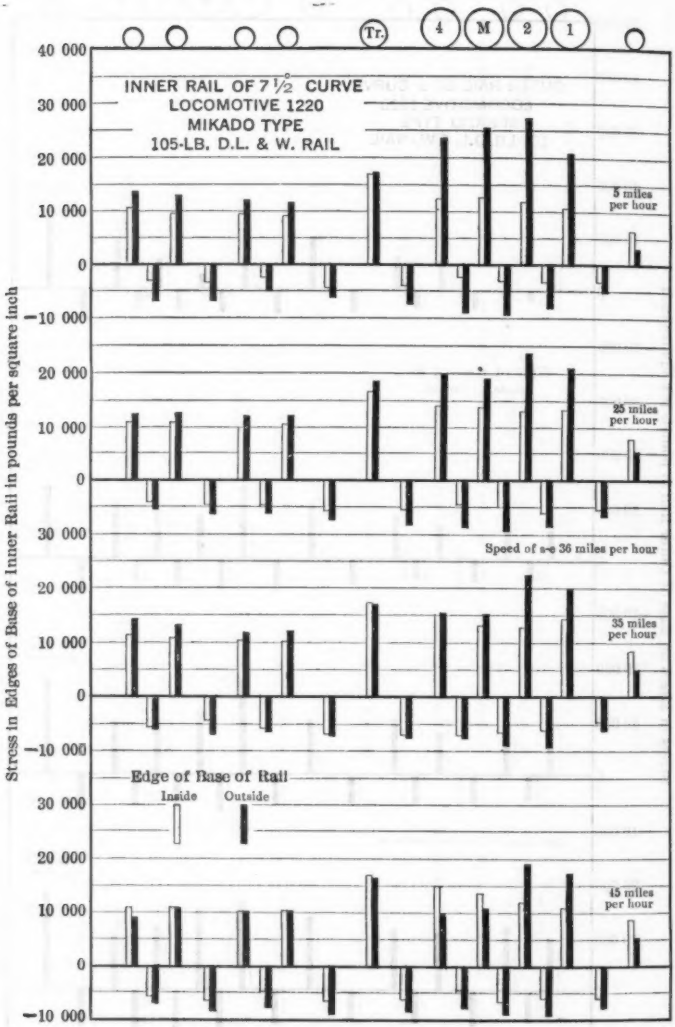


FIG. 56.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER RAIL OF THE 7½° CURVE, SERIES 5170–5190, MIKADO TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

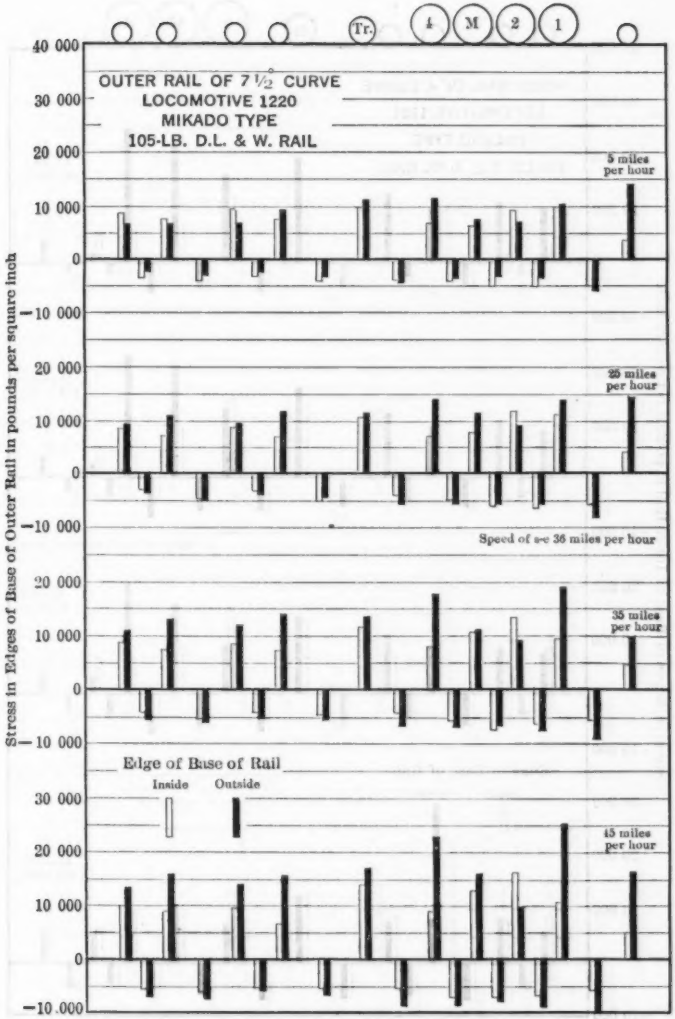


FIG. 57.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE OUTER RAIL OF THE 7½° CURVE, SERIES 5170–5190, MIKADO TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

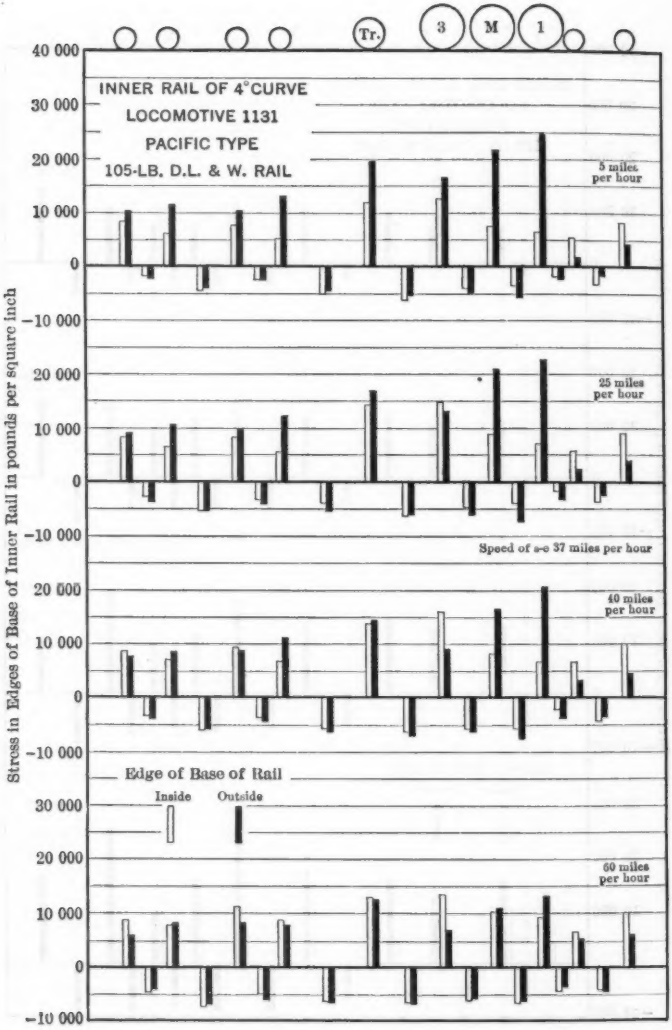


FIG. 58.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER RAIL OF THE 4° CURVE, SERIES 5100–5116, PACIFIC TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

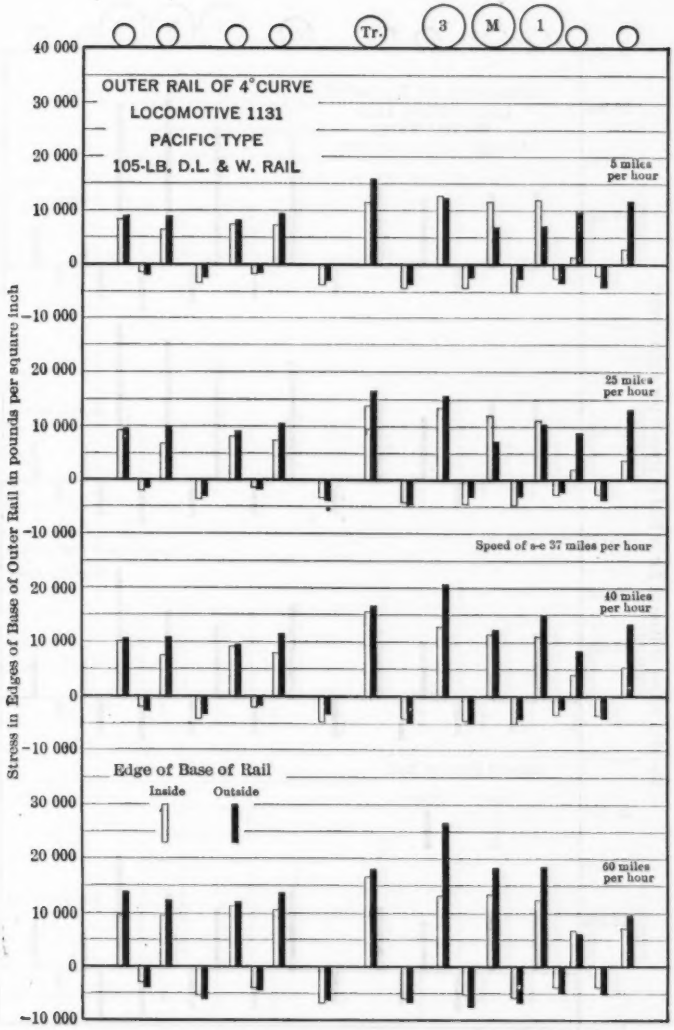


FIG. 59.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE OUTER RAIL OF THE 4° CURVE, SERIES 5100–5116, PACIFIC TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

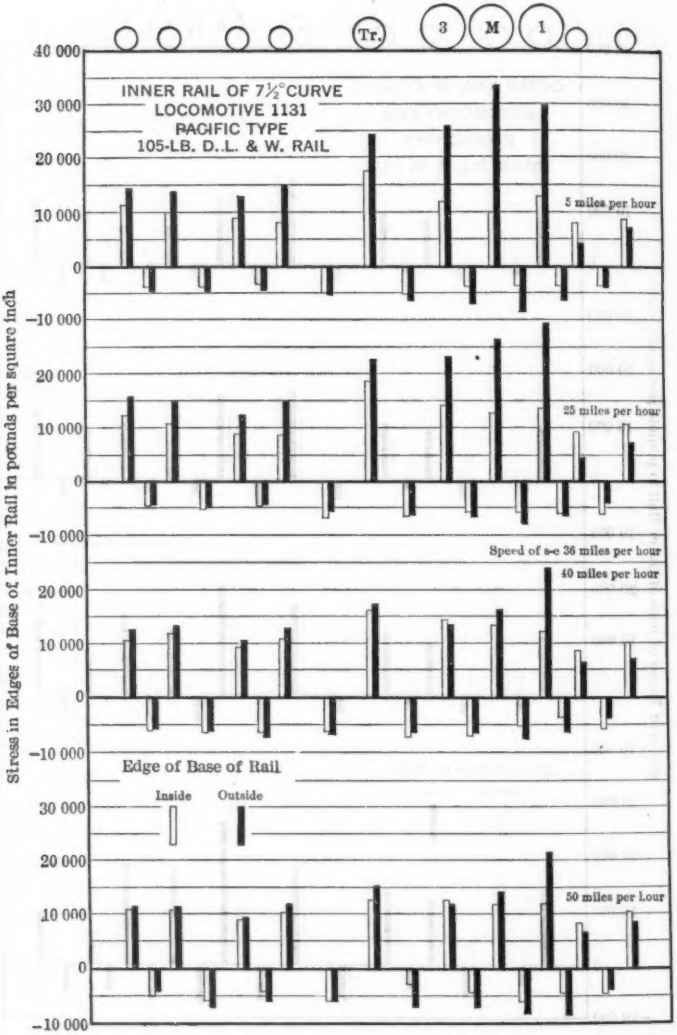


FIG. 60.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER RAIL OF THE 7½° CURVE, SERIES 5117–5135, PACIFIC TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

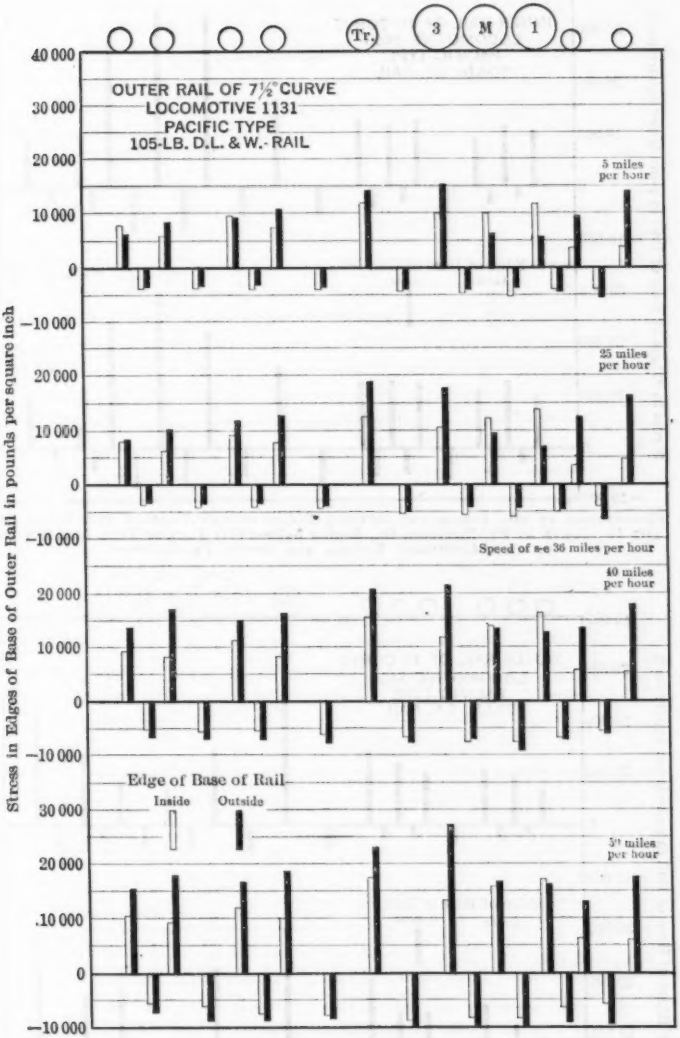


FIG. 61.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE OUTER RAIL OF THE 7½° CURVE, SERIES 5117–5135, PACIFIC TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

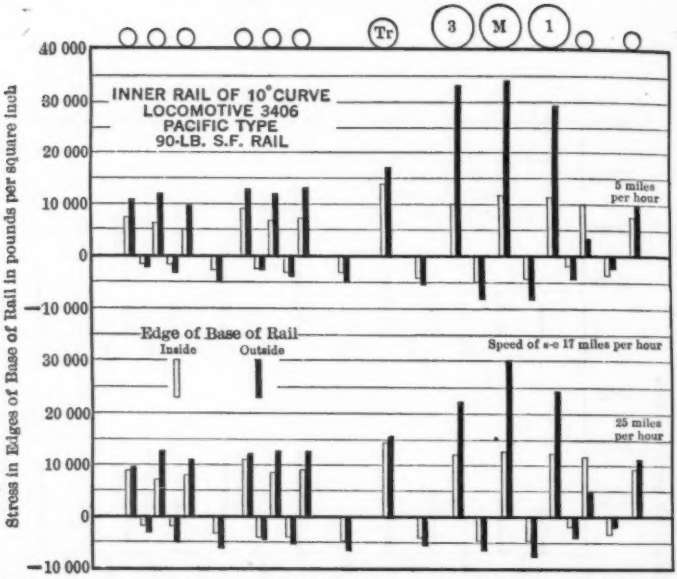


FIG. 62.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER RAIL OF THE 10° CURVE AT FT. MADISON, IA., SERIES 5459–5466, PACIFIC TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

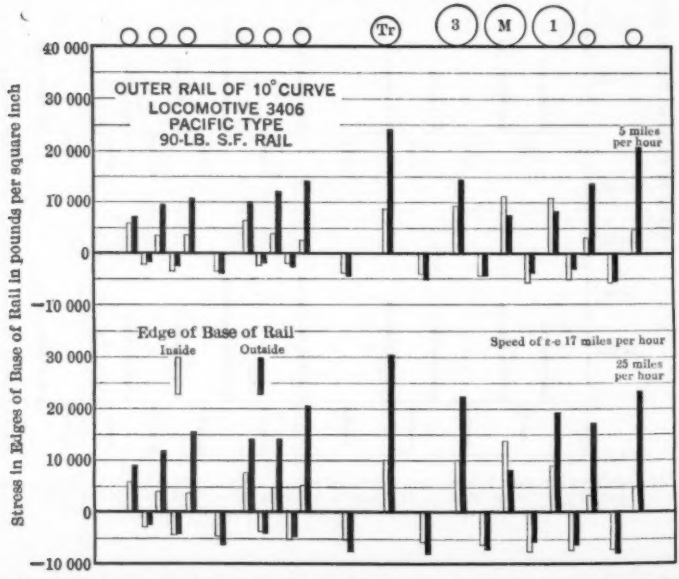


FIG. 63.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE OUTER RAIL OF THE 10° CURVE AT FT. MADISON, IA., SERIES 5459–5466, PACIFIC TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

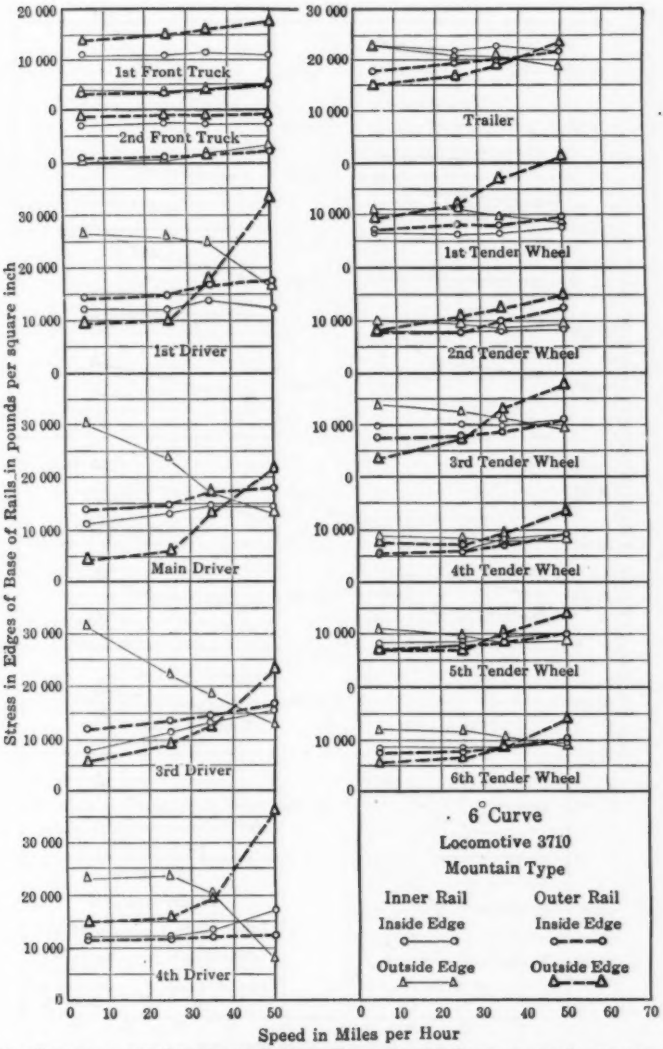


FIG. 64.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER AND OUTER RAILS OF THE 6° CURVE WITH THE MOUNTAIN TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY AT THE SEVERAL SPEEDS.

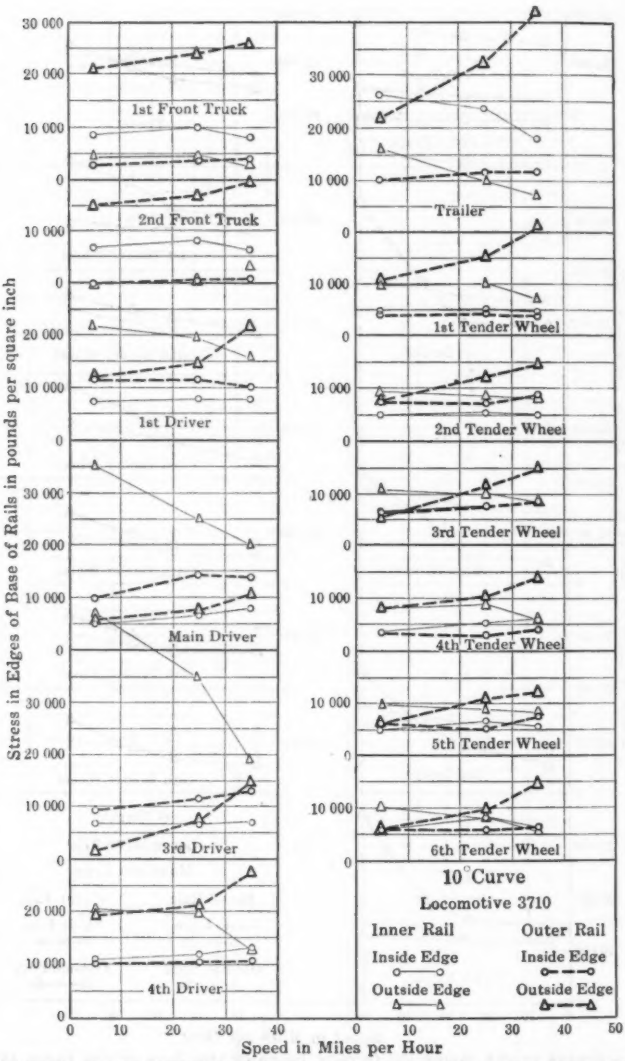


FIG. 65.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER AND OUTER RAILS OF THE 10° CURVE WITH THE MOUNTAIN TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY AT THE SEVERAL SPEEDS.

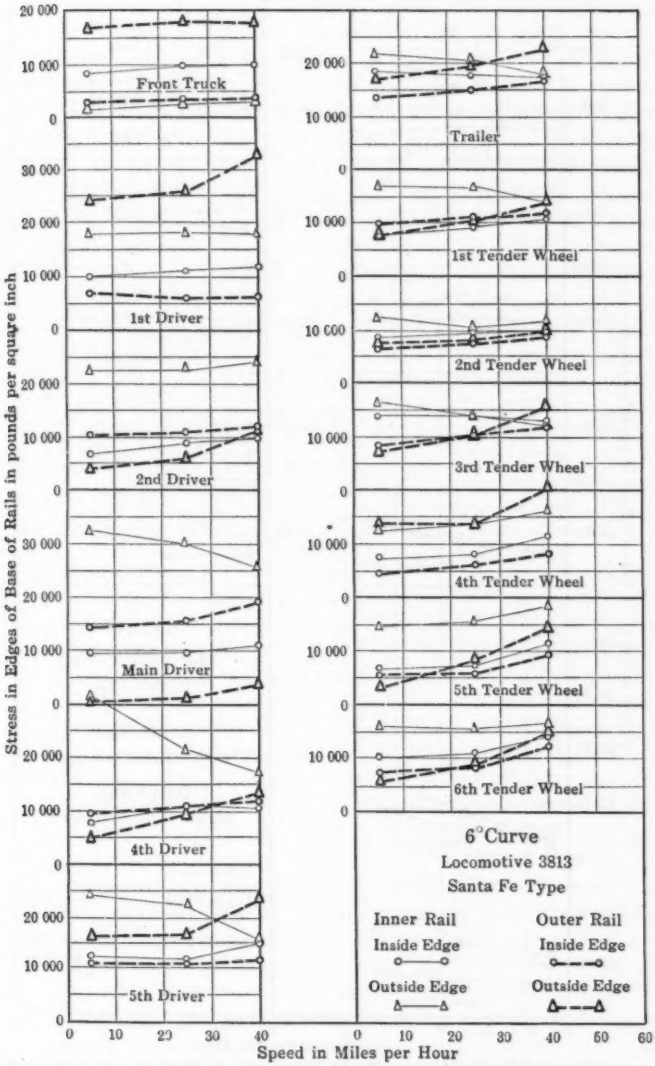


FIG. 66.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER AND OUTER RAILS OF THE 6° CURVE WITH HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY AT THE SEVERAL SPEEDS.

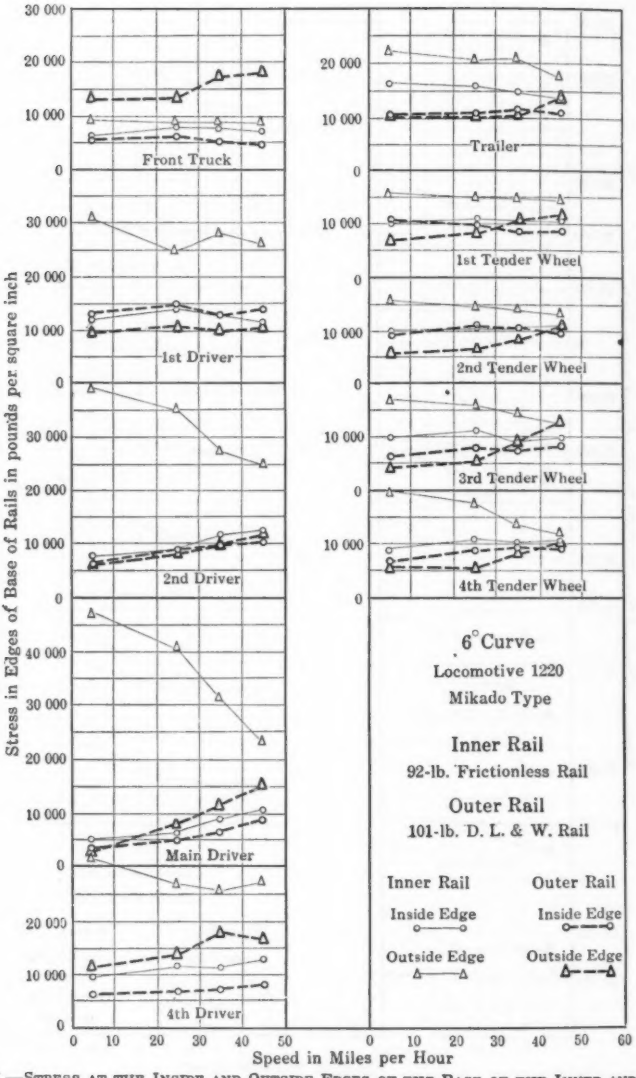


FIG. 67.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER AND OUTER RAILS OF THE 6° CURVE WITH MIKADO TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD AT THE SEVERAL SPEEDS.

tative of the bending of the rail in a vertical plane, or, more strictly, of the bending in a direction normal to the inclination of the track, the two effects being practically identical for the super-elevation of the track used in the tests as the cosine of the angle of inclination is very close to unity. Although stresses in rail are not exactly proportional to the loads when the same total load is differently distributed among the several wheels or differently divided between the two rails, yet, when the differences are not great, the sum of the stresses in the rail under all the wheels will generally not differ greatly for different divisions or distribution of load, and summations of the stresses for each rail may be useful for making comparisons and in checking the action of the locomotive and track. By means of the analytical method given in the first progress report and the solution for unknowns in equations based thereon, the load which will produce the given mean stress may be estimated quite approximately. It will be seen, therefore, that the values of the mean stresses in base of rail (vertical bending stresses) not only will permit comparisons with the stresses in straight track to be made, but will enable the vertical load to be distinguished from the lateral forces or loads that are also developed in passing around the curve. A close estimate may then be made of the distribution of vertical load among the wheels and between the two rails as it is found on the various curves. Two discussions will be given, the first based on the average stresses in the two rails, and the second based on the loads at the individual wheels and the stresses in the rails.

Fig. 68 gives the average of the vertical bending stresses in the inner and the outer rails for the Mountain type locomotive on the 6° curve of the Atchison, Topeka and Santa Fe Railway and also the average vertical bending stress in base of rail found on straight track with the same locomotive. These stresses are the average of those found throughout the revolution of the driver. It is seen that the average of the vertical bending stresses for the inner and outer rail does not differ greatly from the stress found on straight track. Generally speaking, in most of the tests on the curved track, the sum of the vertical bending stresses under all the wheels was nearly that found on straight track, although in some cases it was somewhat larger on the curved track. It is evident that the total of the vertical load effects does not differ greatly in the two kinds of tracks.

It may be interesting to find whether the load is divided between the two rails in the manner indicated by Equations (31) and (32), in Article 12, "The Action of Curved Track". For this purpose, the sum of the vertical bending stresses in base of rail for each of the drivers, trailer, and front truck wheel on one rail was taken, and also the sum of the vertical bending stresses in the base of the other rail. The ratio found by dividing the sum in the outer rail by the average of the two sums for the various tests is given in Tables 21 and 22, as is the corresponding ratio for the inner rail. In subsequent discussion, it is shown that the sum of the stresses is closely representative of the relative loads on the two rails. If the sum of the stresses found on straight track was used as the divisor, the resulting ratios would have quite similar values. The ratios are given in Tables 21 and 22 for the various speeds at which tests

were made. The ratios calculated by Equations (31) and (32) are also given in these tables.

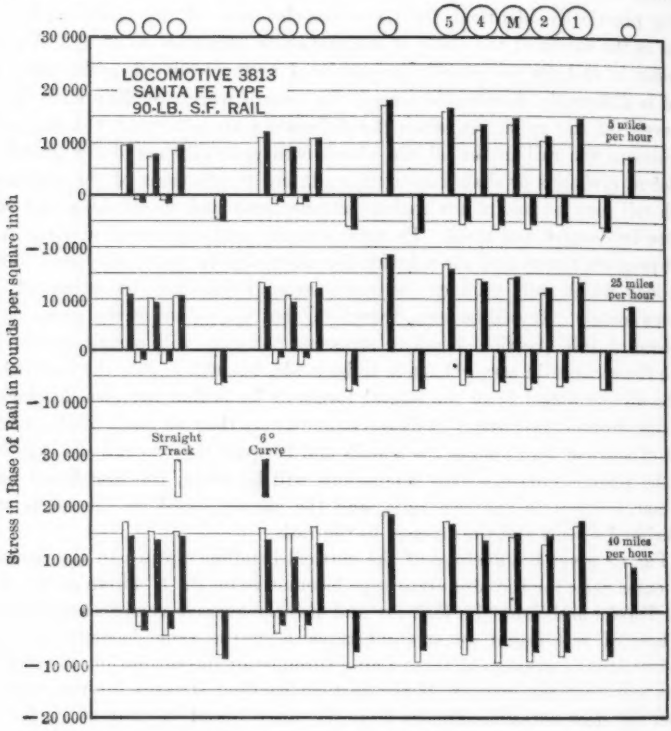


FIG. 68.—VALUES OF THE AVERAGE OF THE STRESSES IN THE INNER AND OUTER RAIL OF THE 6° CURVE AND THE AVERAGE STRESS ON STRAIGHT TRACK, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

For the lower speeds, it is seen that the division of stress between the two rails (and presumably also the division of load) for the Pacific type on the 4° and 7½° curves of the Delaware, Lackawanna and Western Railroad and the Mountain type and Heavy Santa Fe type on the 6° and 10° curves of the Atchison, Topeka and Santa Fe Railway agree very closely with the ratios calculated from the inclination of track and centrifugal force, thus confirming the analysis. The stresses observed under the Mikado type on the 4° curve at the lower speeds give ratios which are much closer to unity than the calculated ratios; it is not known why these differ; for the 7½° curve, the agreement is close. The Pacific type of the Atchison, Topeka and Santa Fe Railway, on the 10° curve, with a super-elevation of 2 in., gives ratios of observed stresses which vary from unity by an amount that is about twice as great as the calculated ratios. It appears from this that, for slight super-elevation, curvature alone gives an effect in changing the division of load between the two rails greater than that

found from the inclination of track and the centrifugal force. Evidently, there is some transfer of load by other means, possibly the action of the equalizing levers.

For the higher speeds, above those corresponding to the speeds of super-elevation, more variation in the results are found. The observed stresses give ratios which correspond to those obtained by analysis in the case of the Mikado type locomotive on the 4° curve, the Pacific type on the 4° curve, the Mountain type on the 6° and 10° curves, and the Heavy Santa Fe type on the 10° curves. Others give seemingly inconsistent ratios at the higher speeds; the Mikado type locomotive on the 7½° curve gives ratios derived from the test values which are larger than those obtained from the analytical values.

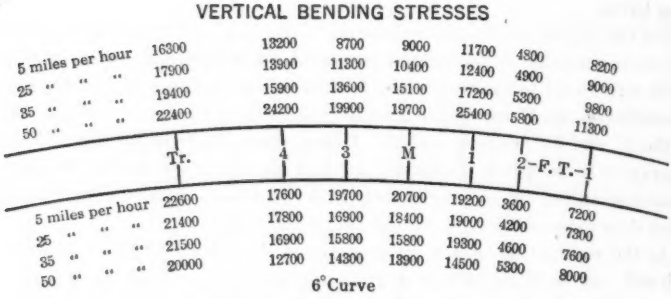
As the summation of the stresses under the wheels of the locomotive for each rail may be taken to represent closely the relative loads on the inner and outer rail, it is evident that at the lower speeds, except for the 10° curve with only 2 in. of super-elevation, the division of the total load between the inner and outer rails is closely represented by the calculated values based on the transverse inclination of track and the centrifugal force. The distribution of load among the several wheels and even its division between the two wheels of one axle are quite different matters, and important variations from the distribution found on straight track will be noted.

Consideration will now be given to the vertical bending stresses under the individual wheels and the loads necessary to produce them.

The vertical bending stress in base of rail under the various wheels of the locomotive (the average stress throughout the revolution of the driver) is recorded in Figs. 69 to 72 for both outer and inner rails for the speeds used in the tests on curved track. Comparison may be made with the stresses under the same wheels in the tests on straight track, as given in Figs. 10 to 20, inclusive, and in Tables 10 to 18, inclusive. In making a study of the stresses, it should be kept in mind that changes in values of the vertical bending stresses at any wheel must be due to changes in the amount of load on the wheel either by itself or in combination with adjoining wheels.

Values of the wheel loads necessary to produce the observed vertical bending stresses may be calculated by the use of the analytical method for determining the stress in rail caused by a group of wheel loads given in the first progress report. An algebraic equation is written for the vertical bending stress at each driver; the observed vertical bending stress at the driver is a known quantity, and the unknown wheel loads of the several drivers are represented as unknown quantities. There will then be as many equations and unknown quantities as there are drivers. The effect of the trailer and the wheels of the front truck is taken into account whenever necessary. The solution of these equations is not troublesome. The fact that the sum of all the loads thus found agrees quite closely with the total load on the drivers should give some confidence in this method of calculating the division of load among the wheels.

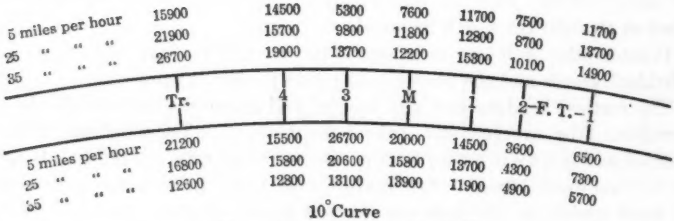
In Table 23 are given the calculated vertical wheel loads on straight track and on the inner and the outer rail of curves for several types of locomotives at a speed of 5 or 10 miles per hour as determined by this method. The results



6° Curve

Mountain Type Locomotive 3710

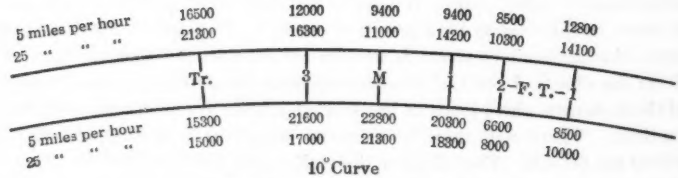
Superelevation 4.7 in. for 35 miles per hour



10° Curve

Mountain Type Locomotive 3710

Superelevation 4.7 in. for 27 miles per hour



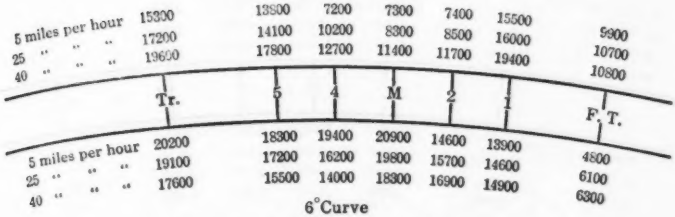
10° Curve

Pacific Type Locomotive 3406

Superelevation 2.0 in. for 17 miles per hour

FIG. 69.—VERTICAL BENDING STRESSES IN BASE OF RAILS OF CURVED TRACK, MOUNTAIN AND PACIFIC TYPE LOCOMOTIVES OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

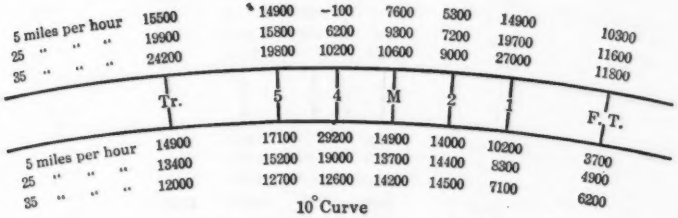
VERTICAL BENDING STRESSES



6° Curve

Heavy Santa Fe Type Locomotive 3813

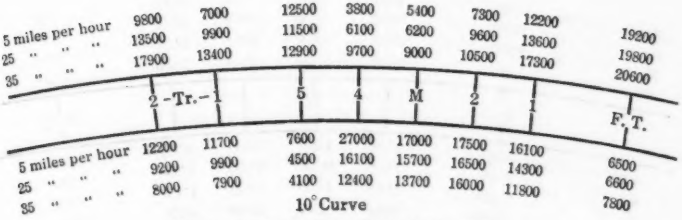
Superelevation 4.7 in. for 35 miles per hour



10° Curve

Heavy Santa Fe Type Locomotive 3813

Superelevation 4.7 in. for 27 miles per hour

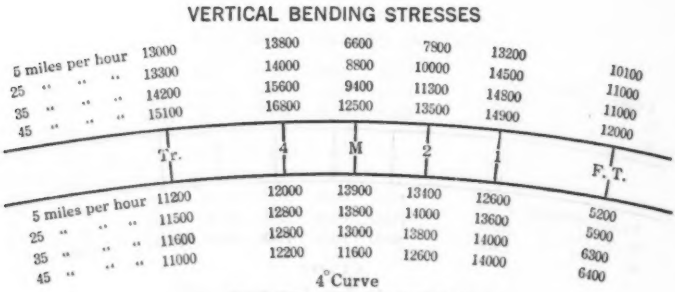


10° Curve

Heavy Santa Fe Type Locomotive 3829

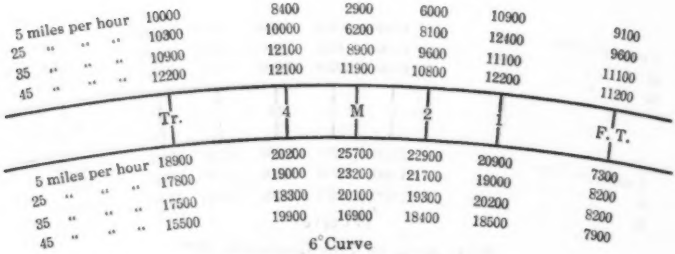
Superelevation 4.7 in. for 37 miles per hour

FIG. 70.—VERTICAL BENDING STRESSES IN BASE OF RAILS OF CURVED TRACK, HEAVY SANTA FE TYPE LOCOMOTIVES OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.



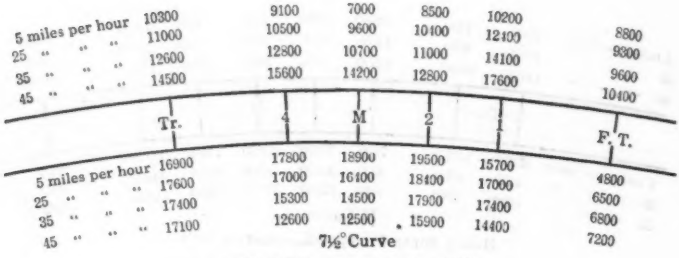
Mikado Type Locomotive 1220

Superelevation 3.7 in. for 37 miles per hour



Mikado Type Locomotive 1220

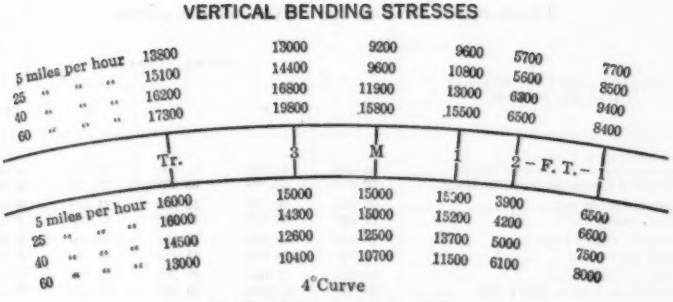
Superelevation 8.5 in. for 46 miles per hour



Mikado Type Locomotive 1220

Superelevation 6.4 in. for 36 miles per hour

FIG. 71.—VERTICAL BENDING STRESSES IN BASE OF RAILS OF CURVED TRACK, MIKADO TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.



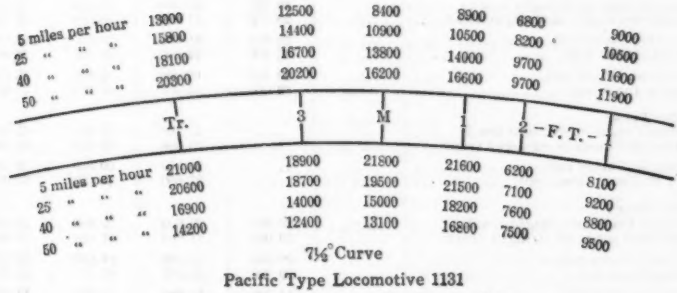


FIG. 72.—VERTICAL BENDING STRESSES IN BASE OF RAILS OF CURVED TRACK, PACIFIC TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

of the calculations were changed proportionally to make the sum of the wheel loads equal the total load on the drivers. The values given in Table 23, although subject to some error, may be taken as representative of the vertical loads transmitted to the rail. The same method, of course, may be applied to any speed.

TABLE 23.—VERTICAL LOADS ON DRIVERS CALCULATED FROM OBSERVED VERTICAL BENDING STRESSES FOR SPEEDS OF 5 AND 10 MILES PER HOUR.

The nominal load on the drivers is also given.

Type of locomotive, degree of curve, and location.	DRIVER NUMBER.				
	5	4	3	2	1
HEAVY SANTA FE TYPE:					
Nominal loads on straight track	32 000	31 200	31 500	29 800	30 200
Calculated loads, on straight track	32 000	33 000	33 000	29 000	28 000
6° curve, Outer rail	25 000	20 000	19 000	21 000	29 000
Ribera, Inner rail	37 000	46 000	47 000	37 000	29 000
10° curve, Outer rail	26 000	8 000	18 000	18 000	28 000
Ribera, Inner rail	41 000	64 000	45 000	37 000	24 000
10° curve, Outer rail	25 000	7 000	21 000	18 000	23 000
Bealville, up grade, Inner rail	35 000	60 000	50 000	37 000	26 000
10° curve, Outer rail	25 000	16 000	22 000	20 000	25 000
Bealville, wet rails, Inner rail	32 000	45 000	44 000	41 000	32 000
10° curve, Outer rail	9 000	21 000	22 000	17 000	24 000
Cajon, up grade, Inner rail	52 000	33 000	47 000	46 000	31 000
10° curve, Outer rail	25 000	16 000	20 000	20 000	27 000
S. P. locomotive, up grade, Inner rail	29 000	50 000	47 000	36 000	27 000
MOUNTAIN TYPE:					
Nominal load on straight track	.....	30 200	30 200	30 400	30 700
Calculated loads on straight track	.....	29 000	31 000	33 000	29 000
6° curve, Outer rail	.....	22 000	21 000	21 000	22 000
Ribera, Inner rail	.....	33 000	42 000	44 000	38 000
10° curve, Outer rail	.....	23 000	16 000	19 000	23 000
Ribera, Inner rail	.....	32 000	53 000	45 000	32 000
PACIFIC TYPE:					
Nominal loads on straight track	.....	.....	30 700	29 200	29 900
Calculated loads on straight track	.....	.....	28 000	32 000	30 000
10° curve, Outer rail	.....	.....	20 000	20 000	19 000
Fort Madison, Inner rail	.....	.....	37 000	45 000	39 000
MIKADO TYPE:					
Nominal loads on straight track	.....	29 600	29 600	29 600	29 600
Calculated loads on straight track	.....	29 000	31 000	31 000	27 000
4° curve, Outer rail	.....	29 000	22 000	24 000	28 000
Dover, Inner rail	.....	29 000	38 000	37 000	30 000
6° curve, Outer rail	.....	14 000	10 000	15 000	19 000
Mt. Tabor, Inner rail	.....	38 000	53 000	50 000	38 000
7½° curve, Outer rail	.....	18 000	18 000	21 000	20 000
Paterson, Inner rail	.....	36 000	45 000	46 000	33 000

It is evident from the diagrams and tables that the changes in vertical bending stresses and in vertical loads at the several drivers are very marked and that important additions to the bending moments in the rail and in the bearing pressures at the top of the rail are found on curves. The increased stress in the inner rail under certain drivers is greatest at the low speed, but even at speeds two-thirds that corresponding to super-elevation increases of stress of 75 to 85% of the increase found at the low speed were common. At the higher

speeds, the special increase in load at the one driver vanishes, and the distribution among the several drivers is much the same as would be expected from a consideration of transverse inclination of track and centrifugal force, except that the vertical bending stress in the outer rail under the outer front driver is very great at the high speeds.

Consider the results for the Heavy Santa Fe type locomotive given in Fig. 70. At 5 miles per hour, the vertical bending stresses in the inner rail of the 6° curve under the third and fourth drivers are much higher than the corresponding ones in the outer rail—far higher than are to be expected from considerations of transverse inclination of track and centrifugal force. In Table 23 it is seen that the loads transmitted by these inner drivers estimated from the observed vertical bending stresses are 47 000 and 46 000 lb., 50% greater than the nominal loads. On the 10° curve at Ribera, at 5 miles per hour, the vertical bending stress in the inner rail under the fourth driver is 29 000 lb. per sq. in. By calculation from the stresses, the load transmitted by this driver is estimated to be 64 000 lb. (see Table 23), more than twice the normal load. The stresses in the outer rail under wheels on the same axles are correspondingly light, the stress under the fourth driver on the 10° curve being zero. Considering the stresses under adjoining wheels, the load on the driver giving a rail stress of zero is calculated to be 8 000 lb., about one-fourth that acting on straight track. For both curves the high stresses in the inner rail become smaller at the higher speeds, and at 35 and 40 miles per hour (above the speed of super-elevation), they are fairly uniform under the several drivers. On the 10° curve, the vertical bending stresses in the outer rail under all the drivers increase with the increase in speed. The stress of 27 000 lb. per sq. in. under the first driver at a speed of 35 miles per hour is the highest vertical bending stress found in the outer rail, and the corresponding calculated load is 47 000 lb.

In the Mountain type locomotive (Fig. 69), the vertical bending stresses in the inner rail of the 6° curve under the second and third drivers at a speed of 5 miles per hour are 2⅓ and 2¼ times those in the outer rail. This means loads of 44 000 and 42 000 lb., 40% more than the normal load on straight track—far more than will be produced by transverse inclination of track alone. On the 10° curve, the corresponding vertical bending stress in the inner rail under the second driver is nearly three times as great as that in the outer rail and under the third driver five times as great. This means loads of 45 000 and 53 000 lb., 50% more than on straight track for the second driver and 75% for the third driver, the latter driver thus giving a very high bearing pressure on the rail. At the speed of 35 miles per hour, the stresses in the two rails of the 6° curve under the various wheels have approached each other and are nearly equal. A somewhat similar condition exists on the 10° curve for the two rails, although the tendency to high stresses in the outer rail under the first and last driver is present with this locomotive, as with the Heavy Santa Fe type. At the speed of 25 miles per hour, the calculated load on the third driver on the inner rail of the 10° curve is 42 000 lb. On the 6° curve, at 50 miles per hour, the vertical bending stress in the outer rail at the first driver is 25 400 lb. per

sq. in. and that at the fourth driver, 24 200 lb. per sq. in. The trailer of the Mountain type gives high stresses in one rail or the other, the highest value being reached in the outer rail of the 10° curve at 35 miles per hour. It is seen that except for the trailer the speed that is higher than that corresponding to the super-elevation gives the most uniform values of the stresses under all the wheels and in the two rails.

The Pacific type locomotive on the 10° curve at Fort Madison, Iowa, (Fig. 69), develops a vertical bending stress in the inner rail under the first driver at a speed of 5 miles per hour that is 2.2 times that in the outer rail, and one under the second driver that is 2.4 times the corresponding stress in the outer rail. The loads necessary to produce these stresses in the inner rail (39 000 and 45 000 lb.) are 1.3 and 1.5 times the normal loads on these two drivers (the driver loads that apply to straight track), although the calculated ratio (see Tables 21 and 22) of load on inner rail to normal load for the super-elevation of 2 in. is only 1.08. At 25 miles per hour (the speed corresponding to the super-elevation is 17 miles per hour), the vertical bending stress in the inner rail under the main driver is still nearly double that in the outer rail and the corresponding calculated loads are 40 000 and 24 000 lb. The distribution of load among the drivers at both speeds is evidently considerably different from that which obtains on straight track. It seems that for this sharp curve and small super-elevation the equalizing system does not act to give an equal distribution of load among the drivers. The Pacific type locomotive on the 4° curve of the Delaware, Lackawanna and Western Railroad, at 5 miles per hour, develops vertical bending stresses in the inner rail under the first and second drivers that are 1.6 times the corresponding stresses in the outer rail. These stresses indicate that the loads on these two drivers, 39 000 and 41 000 lb., are 20 and 25% greater than the normal load, although the calculated ratio (and also the ratio of observed values for the locomotive as a whole) is only 1.15. On the 7½° curve, at 5 miles per hour, the same drivers develop vertical bending stresses in the inner rail 2.4 and 2.6 times those in the outer rail; the corresponding loads on the inner rail are 46 000 and 49 000 lb., 40 to 50% greater than the normal loads, while the ratio calculated from the transverse inclination of the track is only 1.27. On both the 4° and the 7½° curves, at speeds at and slightly above those corresponding to super-elevation, the vertical bending stresses in both rails under all the drivers of this locomotive approach common values, showing that the distribution of load is much the same as that given by the equalizing levers on straight track.

The Mikado type locomotive of the Delaware, Lackawanna and Western Railroad, which has the first and second, and also the third and fourth, drivers connected by equalizing levers, shows a different distribution and grouping of the stresses. The ratios of the vertical bending stress in the inner rail of the 4° curve, at 5 miles per hour, to stress on straight track derived by the analysis and calculation, are in the order of the drivers, 1.04, 1.22, 1.26, and 1.04. For each two drivers that are connected by equalizers, the ratio is higher for the intermediate driver than for the front or rear one, but the average

is close to 1.15, the value calculated from the inclination of track. On the 7½° curve, at 5 miles per hour, the same transfer of load from first to second driver and from fourth to third occurs. The ratios of stress in inner rail to stress in outer rail are 1.5, 2.3, 2.7, and 2.0, respectively. The corresponding loads on the inner rail are 33 000, 46 000, 45 000, and 36 000 lb. These loads show that there is a considerable increase in the bearing pressure on the inner rail over its normal value. For the higher speeds, the stresses in the two rails are much the same for all the drivers. For the 6° curves, the loads on the second and third drivers on the inner rail calculated from the stresses, at 5 miles per hour (see Table 23), are 50 000 and 53 000 lb., respectively, and at 25 miles per hour, 45 000 and 47 000 lb.

It is seen that for all the curves the relation of the average of the vertical bending stresses under all the wheels of a locomotive on the inner rail to the corresponding average of the stresses in the outer rail, taken as a measure of the bending of the rail in a vertical plane, in the main conform fairly closely to what may be expected from the analytical consideration of the transverse inclination of the track and the centrifugal force, as far as the locomotive as a whole is concerned. The principal variation from this conformity to analysis is the Pacific type locomotive on the 10° curve having a super-elevation of 2 in.

The outstanding result of the foregoing study of vertical bending stresses on curved track is the variable distribution of load among the drivers, and, particularly, the greatly increased loads found on some of the drivers beyond that normally expected. These high loads occur principally on the inner rail and are most marked at the low speed, although high values were found in a number of cases at speeds two-thirds that corresponding to the super-elevation. Further examples of high loads under individual drivers at moderate speeds are cited in the discussion of the tests in California (see Article 20, "Results of Tests on 10° Curves in California"). Loads ranging up to 100% in excess of the normal load were found with several of the locomotives. These excessive loads produce high stresses in the rail, but more important still they give very high bearing pressures on the rail. It is a well known fact, of course, that the inner rail of curves suffers greatly from the high pressure on it. The bearing pressures on the rail on straight track constitute a serious problem, and the conditions on the curves seem to be far worse than they have been thought to be. As the bearing pressures found are considerably higher under some of the wheels than those which ordinary analysis of the effect of transverse inclination of track and centrifugal force would predict, it would seem well to have a study made of the action of the equalizing levers and the springs on curved track to try to find the cause of the concentration of load, and to attempt to modify the design so as to obviate such gross inequalities.

15.—Lateral Bending of Rail on Curves.—When forces are applied laterally to the rails of a curve by the wheels of the locomotive (the component forces in the direction of the radius of the curve, either inward or outward, being meant), lateral bending of the rail is produced and a set of stresses is developed which may be termed lateral bending stresses. As the beam strength of the rail in a lateral direction is much less than that which resists bending in a

vertical direction (the section modulus, $\frac{I}{c}$, about a vertical axis being only about one-fifth of that about a horizontal axis), large lateral bending stresses will be developed by a lateral bending moment relatively much smaller than the vertical bending moment that causes vertical bending stresses of the same magnitude. The stress in any fiber, of course, will be the sum of the fiber stress due to vertical bending and that due to lateral bending, the nature of the stress either tension or compression being taken into consideration. It will help in understanding the cause of the lateral bending and in applying the analysis given in Article 12, "The Action of Curved Track", if something is known of the position and direction taken by the wheels of the locomotive in traversing a curve.

In Figs. 73 and 74 are shown the relative positions of the wheels of five locomotives on several curves, the dimensions are given in inches and represent the distance from the gauge side of the rail to the flange of the wheel. The measurements were made with a view of obtaining a general notion of the position and direction taken by the wheels, but no effort was made to secure great accuracy or completeness in the tests and measurements. The results, however, may be taken to be fairly accurate. The locomotive was run over the track at slow speed and allowed to come to rest without the application of the brakes. The test was not repeated. On the Atchison, Topeka and Santa Fe Railway, measurements were made between the gauge side of the rail and the flange of the wheel. A steel scale was used and notwithstanding the difficulties of measurement the data are probably accurate to $\frac{1}{16}$ in. On the Delaware, Lackawanna and Western Railroad, no measurements were made except for the last diagram of Fig. 74, but observations were taken to see whether the flange bore on the side of the rail, or was away from it a distance less than or more than $\frac{1}{2}$ in. The radial line on Figs. 73 and 74 is drawn at the position where the radius of the curve is perpendicular to the line taken by the wheel base, the determination being made on a large scale drawing. The point at which this radial line crosses the inner rail may be taken to be the probable center of rotation of the locomotive frame. Generally speaking, the flange of the outer front driver bears against or is close to the outer rail, and a line along the gauge side of the flanges of the drivers cuts the outer rail at the outer front driver or somewhat in front of it. The flange of the outer rear driver does not touch the outer rail and in several cases the flange of the inner rear driver is closer to the rail than that of the outer one The flange of the inner front driver is well away from the rail. Generally, too, the flange of the driver next to the rear one is close to the inner rail, although it does not bear against it as the outer front driver bears against the outer rail. The center of rotation (if this is the center of rotation) is the point at which the radial line marked on the diagrams crosses the inner rail. For the locomotives having eight or ten drivers, this point is located on the inner rail at the driver next to the rear driver. Locomotive No. 3710, on the 10° curve, appears to be an exception to this; the point is here found under the second driver. In the case of the Pacific type locomotive, one diagram shows

the point to be at the rear driver and one between the second and third driver. The former has the front driver flange against the outer rail; in the second one, it is well away from it.

The flange of the outer wheel of the front truck generally bears against the outer rail. In the case of the four-wheel leading truck, both outer wheels have this bearing. The flanges of the trailers do not touch the outer rail. In one case only do they touch the inner rail. They take a position away from the two rails sometimes closer to one and sometimes to the other. The first wheel of the trucks of the tender generally bears against the outer rail and the line of the wheel base is a secant line cutting the outer rail at this point. In some cases, the front truck does not take this position; its location seems to be somewhat variable. For those trucks having the front wheel bearing against the outer rail, the center of rotation seems to be at or near the rear wheel of the truck.

The positions given in Figs. 73 and 74 may be taken to be representative of the position of the wheels at low speeds. For higher speeds, some modifications may be expected. The tendency will be for the wheels to move somewhat toward the outer rail, although it seems evident that generally there will not be much change in position with respect to the two rails. There will be, of course, an increase of the pressure of the flange of the front wheel against the rail and also a change in the amount of lateral pressure transmitted by means of friction between the wheel and the rail, but these changes may not be expected to result in any marked change in the relative position of the wheels and rails. This conclusion is borne out by the nature and amount of the lateral bending stresses found in the tests.

In Figs. 75 to 78, inclusive, is recorded the magnitude of the lateral bending stresses in the outer and inner rails under the several wheels of the locomotive and tender at the speeds at which tests were made, the stress being given in pounds per square inch, for the tests at Ribera, N. Mex., Fort Madison, Iowa, and Dover, Paterson, and Mount Tabor, N. J. This lateral bending stress is the stress by which the stress at one edge of the base of the rail exceeds that at the middle of the base of rail; its magnitude is found by taking one-half the difference between the stresses at the inside and outside edges of the base of rail. The sign in front of the stress is indicative of the nature of the lateral bending of the rail; a positive sign indicates that the bending acts to increase the curvature of the rail, the negative that the bending tends to straighten the rail. The signs apply alike to both outer and inner rail. A positive bending in the outer rail at the point of contact of a wheel implies an outward thrust against the rail at the given point; a positive bending in the inner rail implies that a lateral pull on this rail acts toward the outer rail. The diagrams give important information on the presence and nature of lateral forces and the manner of the lateral bending in the rail. Some of the outstanding features may be noted, as follows:

The lateral bending in the outer rail, under the wheels of the front truck, is found to be always positive (indicating an outward thrust) and it is generally positive also in the inner rail. This is true at all speeds. It indicates

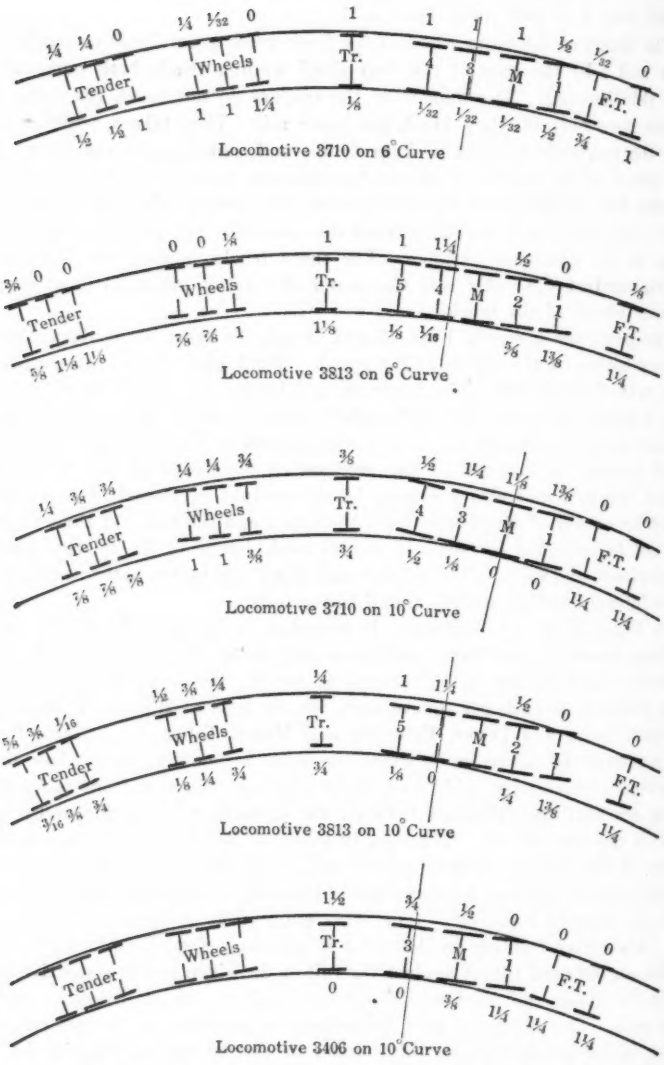
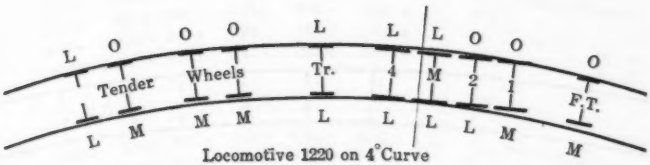
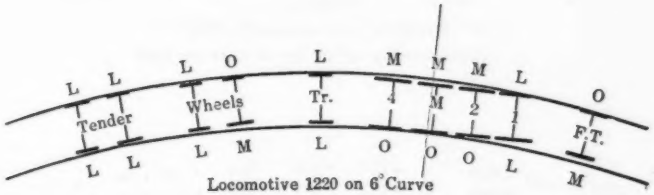


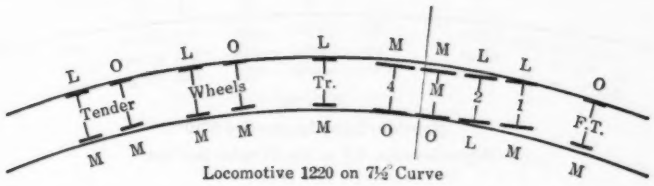
FIG. 73.—RELATIVE POSITION OF WHEELS OF LOCOMOTIVES AND TENDERS WITH RESPECT TO THE RAILS OF CURVED TRACK ON THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.



Locomotive 1220 on 4° Curve



Locomotive 1220 on 6° Curve

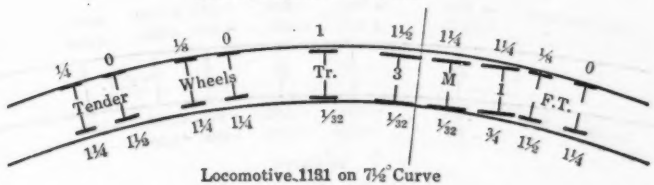


Locomotive 1220 on 7½° Curve

Key for 1220

O-Elange Touching Rail L-Small Space-Up to ½ inch

M-Large Space-Over ½ inch



Locomotive 1181 on 7½° Curve

FIG. 74.—RELATIVE POSITION OF WHEELS OF LOCOMOTIVES AND TENDERS WITH RESPECT TO THE RAILS OF CURVED TRACK ON THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

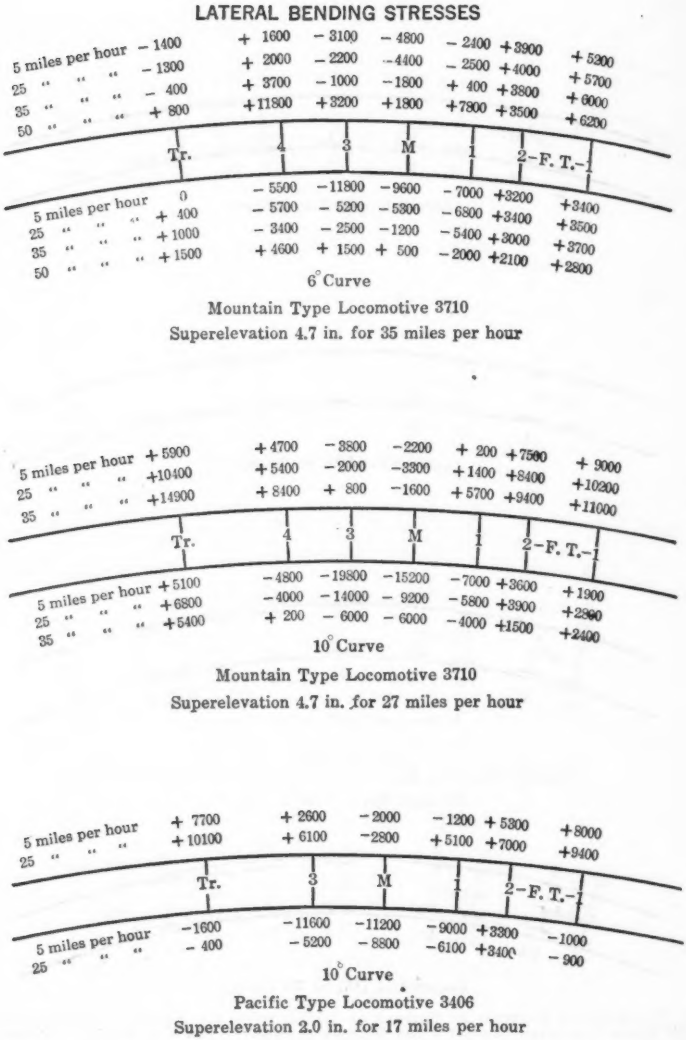
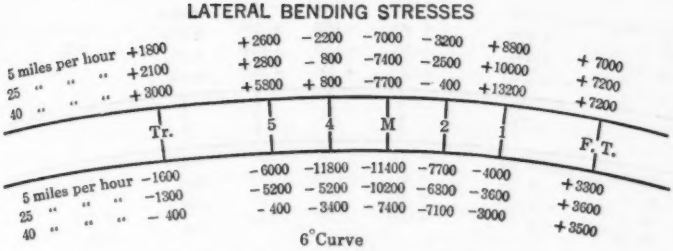
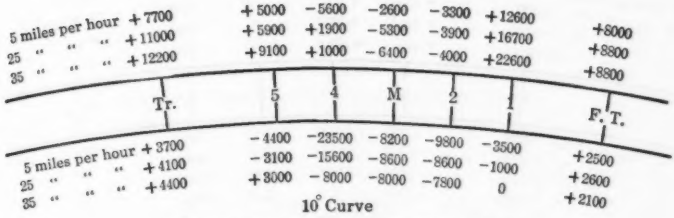


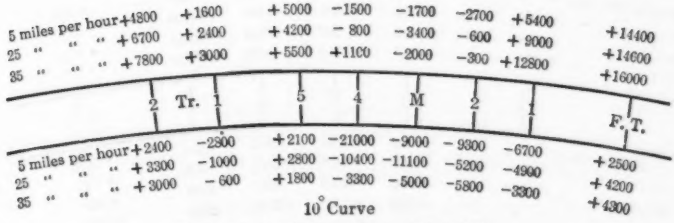
FIG. 75.—LATERAL BENDING STRESSES IN BASE OF RAILS OF CURVED TRACK, MOUNTAIN AND PACIFIC TYPE LOCOMOTIVES OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.



Heavy Santa Fe Type Locomotive 3813
Superelevation 4.7 in. for 35 miles per hour

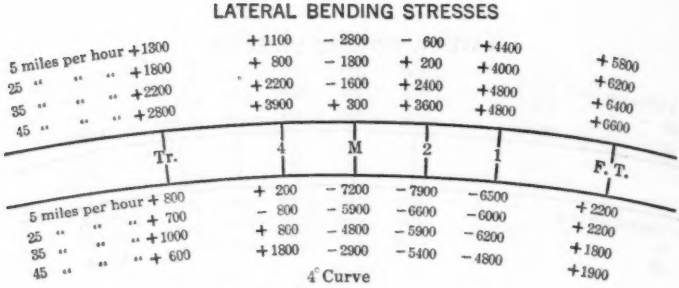


Heavy Santa Fe Type Locomotive 3813
Superelevation 4.7 in. for 27 miles per hour



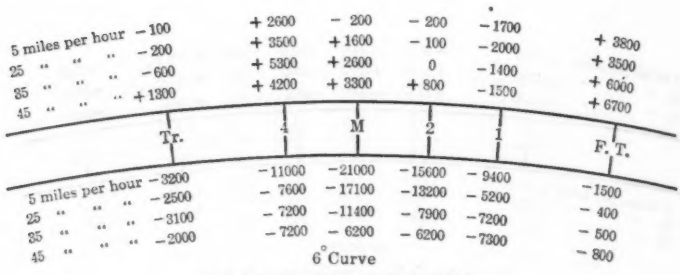
Heavy Santa Fe Type Locomotive 3829
Superelevation 4.7 in. for 27 miles per hour

FIG. 76.—LATERAL BENDING STRESSES IN BASE OF RAILS OF CURVED TRACK, HEAVY SANTA FE TYPE LOCOMOTIVES OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.



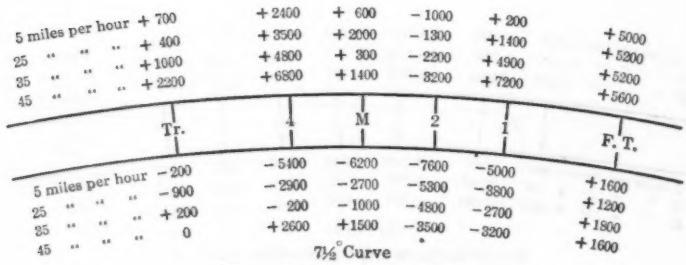
Mikado Type Locomotive 1220

Superelevation 3.7 in. for 37 miles per hour



Mikado Type Locomotive 1220

Superelevation 8.5 in. for 46 miles per hour



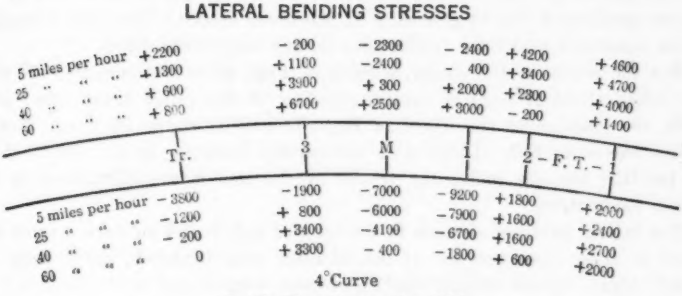
Mikado Type Locomotive 1220

Superelevation 6.4 in. for 36 miles per hour

FIG. 77.—LATERAL BENDING STRESSES IN BASE OF RAILS OF CURVED TRACK, MIKADO TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

that the flanges of the outer wheel of the front truck always have an important part in changing the direction of the locomotive.

In the locomotive having two-wheel front trucks, the lateral bending of the outer rail at the outer front driver is positive at all speeds, with the exception of the Mikado type on the 6° curve, where the super-elevation is 8.5 in. With the locomotives having four-wheel leading trucks, the bending at the outer front driver is generally negative at the low speed, becoming positive at a higher speed, with the exception that it remains negative in the case of the



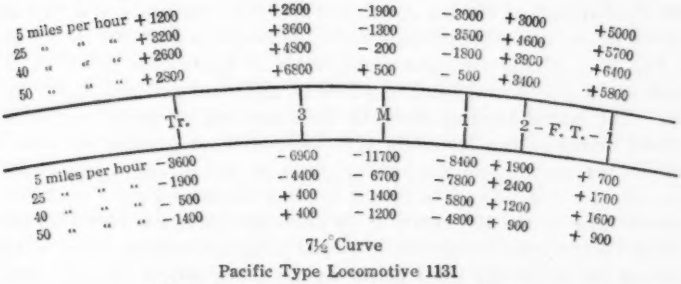


FIG. 78.—LATERAL BENDING STRESSES IN BASE OF RAILS OF CURVED TRACK, PACIFIC TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

Pacific type locomotive on the 7½° curve. The nature of the bending of the outer rail is in agreement with what is to be expected from the position of the outer front driver given in Figs. 73 and 74.

The outer rear driver gives a positive lateral bending in the outer rail (outward thrust on the rail) at all speeds, the one exception being the Pacific type locomotive on the 4° curve at a speed of 5 miles per hour.

At low speeds, all the other drivers generally give negative bending in both inner and outer rail, tending to straighten the rail. The principal ex-

ception is that the bending moment under the inner fifth driver of the Santa Fe type locomotive is sometimes positive.

An increase of speed results in an increase of lateral bending in the outer rail, decreasing the value of the lateral bending stress if the bending is negative and increasing the stress if it is positive.

An increase of speed results in a decrease in the lateral bending in the inner rail, the numerical value of the lateral stress decreasing when the bending moment is negative and increasing when it is positive.

The trailer has an outward thrust on the outer rail in every case at the higher speeds and almost generally at the lower speeds. The lateral bending of the inner rail under the trailer may be one way or the other.

For the wheels of the tender, a study of Figs. 42 to 63, inclusive, will show that when there is positive lateral bending at the outer front wheel of a truck, the position of the wheel in Figs. 73 and 74 shows its flange bearing against the outer rail. There is no uniformity, however, in the way in which the bending stresses under the trucks of the tender are distributed in the various locomotives.

The lateral bending stresses in the base of rail found on curved track and shown in these diagrams are of considerable magnitude—in many cases extremely high. Severe lateral bending stresses were found in the inner rail of the 10° curve under the fourth driver of the Santa Fe type locomotive at the low speed. The value given on one of the diagrams is 23 500 lb. per sq. in. In the tests at Bealville, Calif., which will be described in Article 20, "Results of Tests on 10° Curves in California", the lateral bending stress under the fourth driver of the Santa Fe type locomotive of the Atchison, Topeka and Santa Fe Railway, at the low speed, was 25 800 lb. per sq. in. and that under the same driver of the Southern Pacific locomotive was 20 300 lb. per sq. in. (see Fig. 96). Under the outer front driver of Locomotive No. 3813, at the highest speed, the lateral bending stress in the outer rail was 22 600 lb. per sq. in. The lateral bending stress in the inner rail of the 10° curve, under the third driver of the Mountain type locomotive, at 5 miles per hour, was 19 800 lb. per sq. in., and the similar stress on the 6° curve was 11 800 lb. per sq. in. (see Fig. 75). The lateral bending stress under the trailer of the Mountain type on the 10° curve, at 35 miles per hour, was 14 900 lb. per sq. in. The Pacific type locomotive developed a lateral bending stress of 11 600 lb. per sq. in. under the third driver in the inner rail of the 10° curve at Fort Madison, at 5 miles per hour, and of 10 100 lb. per sq. in. in the outer rail under the trailer, at a speed of 25 miles per hour (see Fig. 75). The Mikado type locomotive developed a lateral bending stress of 21 000 lb. per sq. in. in the base of the inner rail of the 6° ourve under the third driver and 15 600 lb. per sq. in. under the second driver, these stresses being developed at slow speeds on track having a super-elevation of 8.5 in. (see Fig. 77). On the 7½° curve, with a super-elevation of 6.4 in., the corresponding lateral bending stresses were 6 200 and 7 600 lb. per sq. in., respectively (see Fig. 77). On the 4° curve, the corresponding lateral bending stresses were 7 200 and 7 900 lb. per sq. in. The Pacific type locomotive of the Delaware, Lack-

awanna and Western Railroad developed a lateral bending stress of 9 200 lb. per sq. in. in the inner rail under the first driver, on the 4° curve, and on the 7½° curve, 11 700 lb. per sq. in. under the second driver; even on a 4° curve, it is seen that the lateral bending stress under some of the wheels is considerable in comparison with the vertical bending stress in the rail under these same wheels (see Fig. 78).

When the stress due to lateral bending is combined with that due to vertical bending, it will be seen that the resulting stress may be very great (see Figs. 42 to 67). This is the more marked because the high lateral bending stresses generally occur under wheels giving very high vertical bending stresses, much greater than those developed on straight track. Thus, for example, at the outside edge of the base of the inner rail of the 10° curve, under the fourth driver of the Santa Fe type locomotive, at 5 miles per hour, the stress is the sum of 23 500 and 29 200, resulting in a stress of 52 700 lb. per sq. in. This value is the average of a large number of runs; not infrequently stresses several thousand pounds greater than this were found, indeed, one of 75 000 lb. per sq. in. was recorded. It may be added that the value of the lateral bending stresses under many of the wheels ranged from 40 to 100% of the vertical bending stress under the same wheel.

The lateral bending stresses observed at points between wheels are of measurable magnitude. Their values bear somewhat the same relation to the stresses under the wheels as that found with the vertical bending stresses on straight track. In general, the direction of the bending at points between wheels is opposite in character to that at the wheels, positive bending if that at the wheels is negative and negative bending if the other is positive. This inward and outward bending corresponds to the positive and negative bending moments found ordinarily at and between wheels for vertical load alone. The torsional stresses developed have not been analyzed.

16.—Lateral Bending Moments.—The lateral resisting moment developed in the rails, which, of course, at any point is equal to the lateral bending moment, may be readily calculated from the observed lateral bending stress by the use of the lateral section modulus, $\frac{I}{c}$, of the rail. The variation in the lateral bending moment at and between wheels throughout the length of the locomotive and at the several speeds offers an interesting view of the lateral bending action of the rails of curved track as the load passes. Figs. 79 to 83, inclusive, show the lateral bending moment in the outer and inner rails for four types of locomotives on several curves of different degrees of curvature. As before, positive bending is taken to be that which increases the curvature of the rail, and negative bending that which tends to straighten the rail.

A study of these diagrams brings out interesting characteristics of the different locomotives. The Mountain type on the 6° curve (Fig. 79) has much the same characteristics as it has on the 10° curve, although on the 6° curve, except at low speed, the lateral bending moments are small with the exception of the outer rail under the fourth driver. Except at the speed of 50 miles per hour, the principal turning action is given by the wheels of the front truck; on

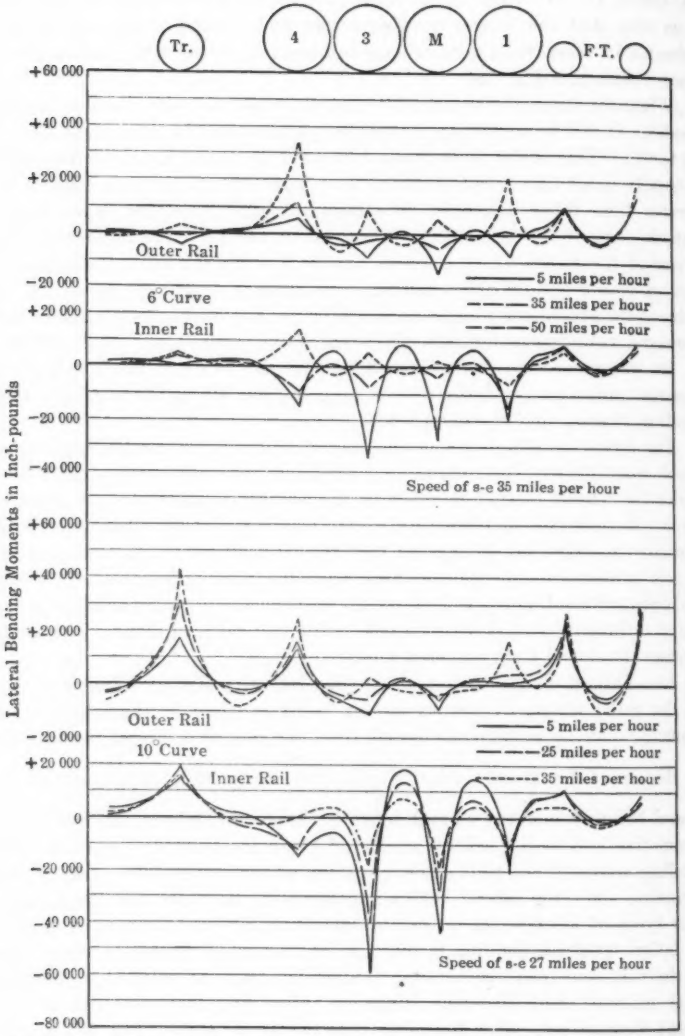


FIG. 79.—LATERAL BENDING MOMENTS IN OUTER AND INNER RAILS OF CURVED TRACK WITH MOUNTAIN TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

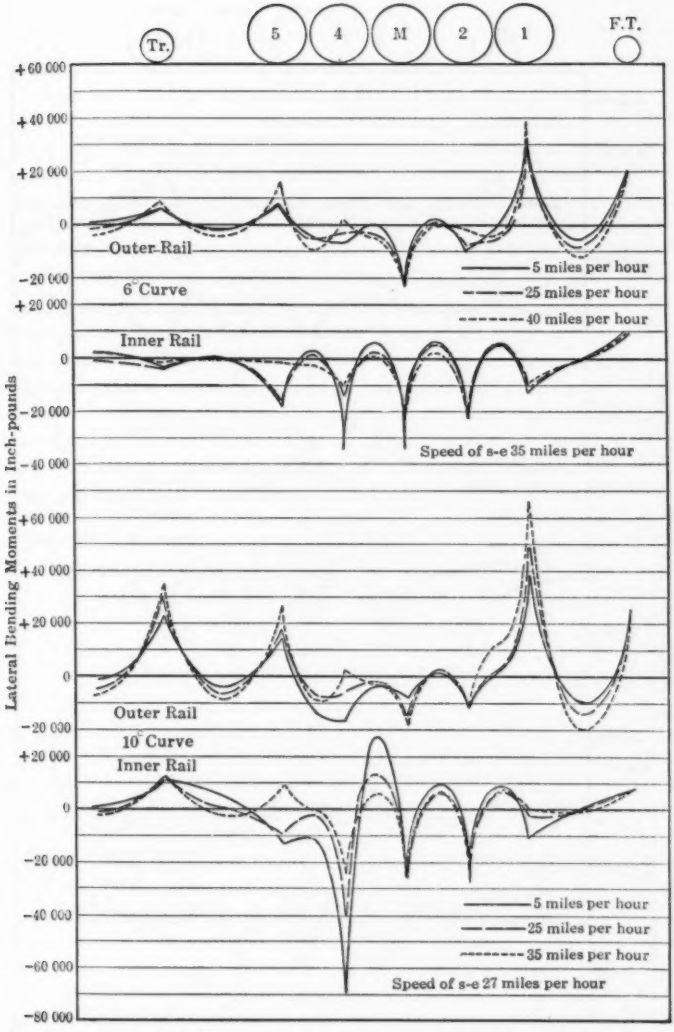


FIG. 80.—LATERAL BENDING MOMENTS IN OUTER AND INNER RAILS OF CURVED TRACK WITH HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

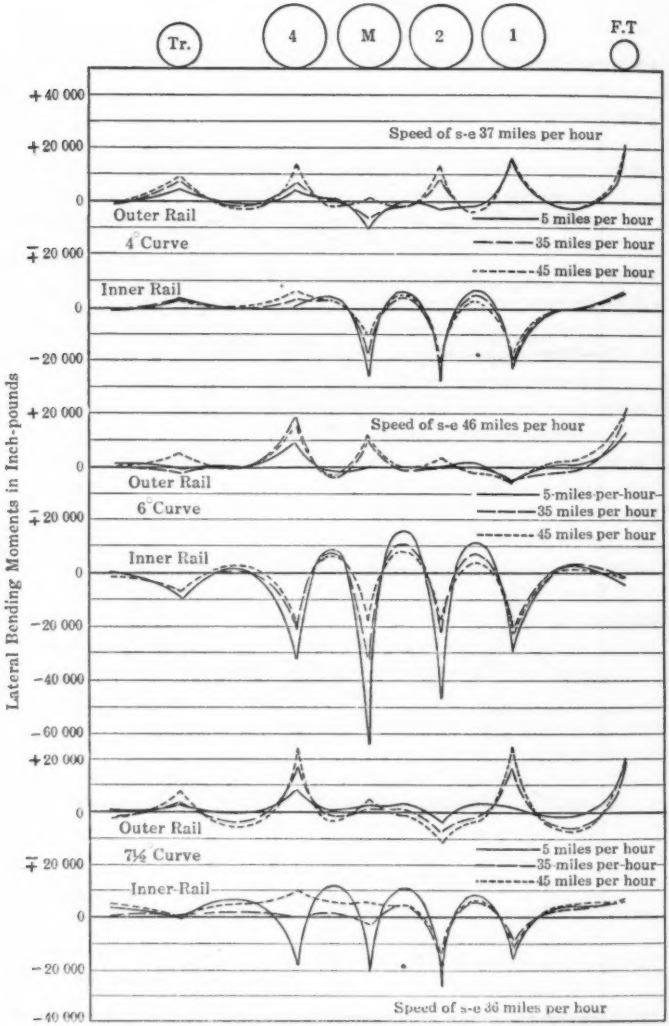


FIG. 81.—LATERAL BENDING MOMENTS IN OUTER AND INNER RAILS OF CURVED TRACK WITH MIKADO TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

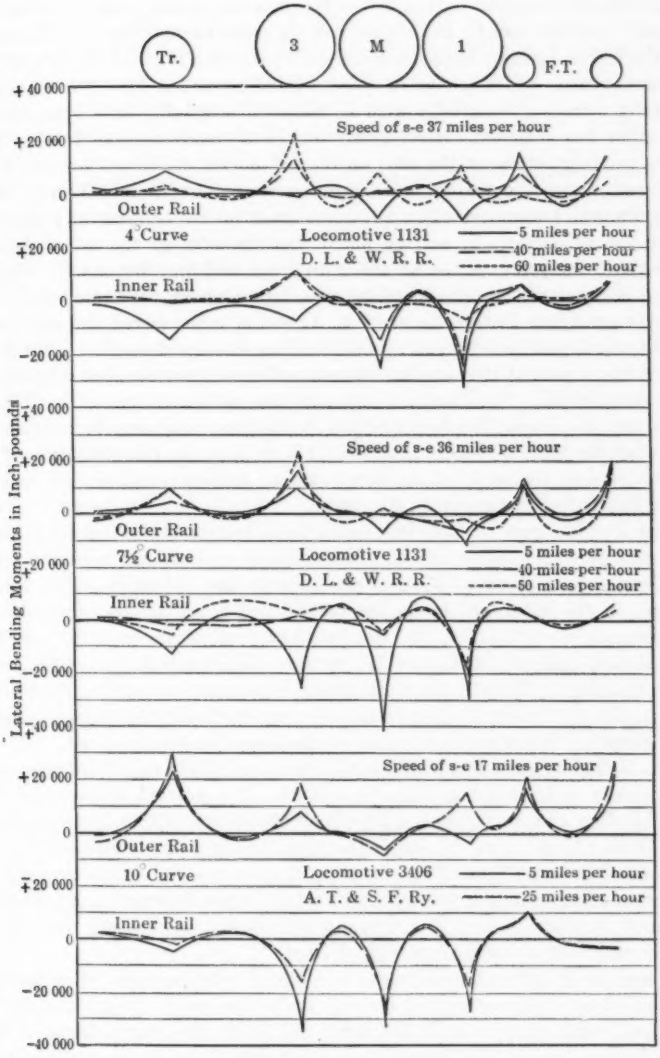


FIG. 82.—LATERAL BENDING MOMENTS IN OUTER AND INNER RAILS OF CURVED TRACK WITH PACIFIC TYPE LOCOMOTIVES.

both curves at the high speed, the front outer driver also participates in the turning action. With the Santa Fe type locomotive (Fig. 80), the magnitude of the bending moment on the 10° curve is quite marked, and it varies considerably with the speed. In this, as with the other locomotives, it will be noted that the lateral bending moment developed by the wheels of the front truck does not vary greatly with a change in speed, while that under the outer front driver generally increases materially with an increase in speed. It is apparent that the trailer has sufficient lateral swing on the 6° curve to give freedom from lateral bending action on the rail; on the 10° curve, the lateral bending in the outer rail under the trailer is very noticeable. The Double Trailer Heavy Santa Fe type locomotive (Fig. 83) gives small lateral bending at points back of the fourth driver. It will be noted that the sum of the lateral bending moments in the outer rail under the front truck and first driver with this locomotive is equal to the corresponding sum found under the same wheels with the ordinary Santa Fe type locomotive. It will be remembered that the locomotive diagrams and the observed vertical bending stresses show that in the double trailer type a part of the load has been transferred from the first driver to the front truck wheel. The Pacific type locomotive on the 10° curve (see Fig. 82) shows that the lateral bending moment in the inner rail may be considerable, even when the super-elevation is small. It is evident also that the trailer of this locomotive does not have full freedom to swing, but bears against the outer rail as the locomotive is turned on the curve.

The Pacific type of the Delaware, Lackawanna and Western Railroad (Fig. 82) shows a bending moment in the 105-lb. rail of only moderate magnitude, except at a speed of 5 miles per hour on the 7½° curve. The bending moment in the inner rail under the second and third drivers of the Mikado type locomotive on the 6° curve, at a speed of 5 miles per hour, is much more marked than that on the 4° and 7½° curves. The stress due to vertical bending in the inner rail under the main driver is not much greater in the 6° curve than in the 7½° curve, so that it may be expected that the large amount of super-elevation is not the only cause of the large bending moment.

The lateral bending moments for the outer and inner rails have been added algebraically and plotted in Figs. 84 and 85. The general trend of the combined bending moments under and between the several wheels is very striking. Figs. 79 to 83, inclusive, illustrate the large bending action developed in the two rails separately; these diagrams show the intense and severe lateral bending action which comes on the track as a whole as the locomotive passes around the curve, tending to push the ties and ballast first to one side and then to another and to throw the curve out of line.

It may be of interest to estimate this lateral bending moment in terms of the vertical bending moment of the rail under the same wheel. The ratio of the lateral bending moment in the inner rail under the fourth driver of three Santa Fe type locomotives (two of them belonging to the Atchison, Topeka and Santa Fe Railway and one to the Southern Pacific Company) to the vertical bending moments developed at the same time averages 0.16 at the lowest speed on the 10° curves. It should be recalled too that the vertical bending stress under this driver was far greater than the normal and that the pressure of the

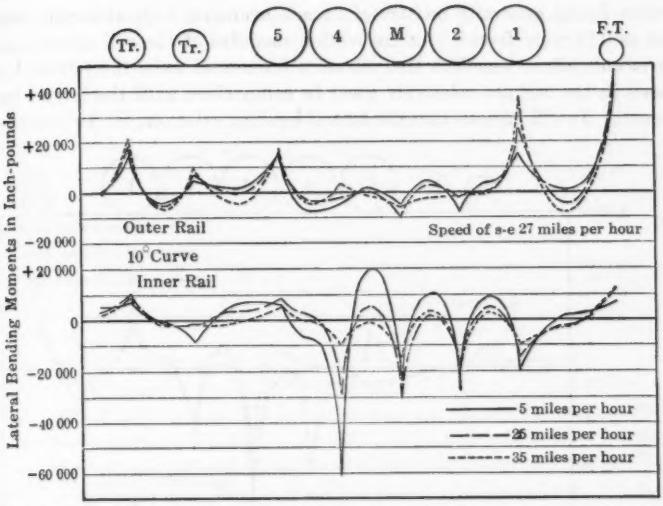


FIG. 83.—LATERAL BENDING MOMENTS IN OUTER AND INNER RAILS OF CURVED TRACK WITH DOUBLE TRAILER HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

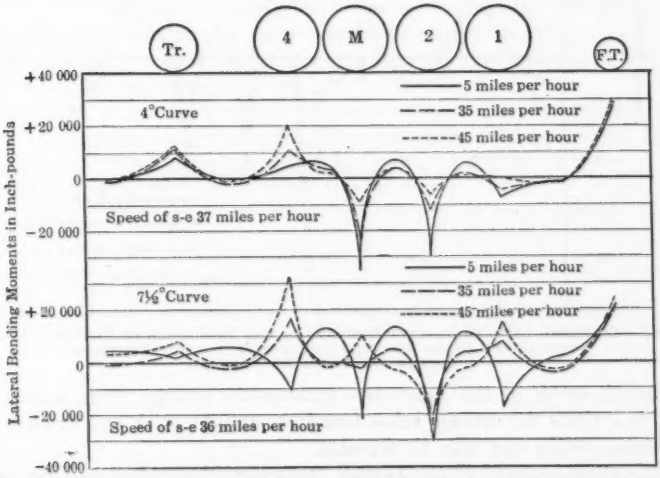


FIG. 84.—COMBINED LATERAL BENDING MOMENTS IN THE TWO RAILS OF CURVED TRACK WITH THE MIKADO TYPE LOCOMOTIVE OF THE DELAWARE, LACKAWANNA AND WESTERN RAILROAD.

driver on the rail must have been nearly twice the normal load. The ratio of the lateral bending moment under the same wheels to the vertical bending moment found generally on straight track averaged 0.04, although values as great as 0.14 were found. As the section modulus of the rail about a vertical axis is only about one-fifth that about a horizontal axis, the lateral bending stresses in the rail are relatively great in comparison with the lateral bending moments. It will be seen that the lateral bending effect on the 10° curve under

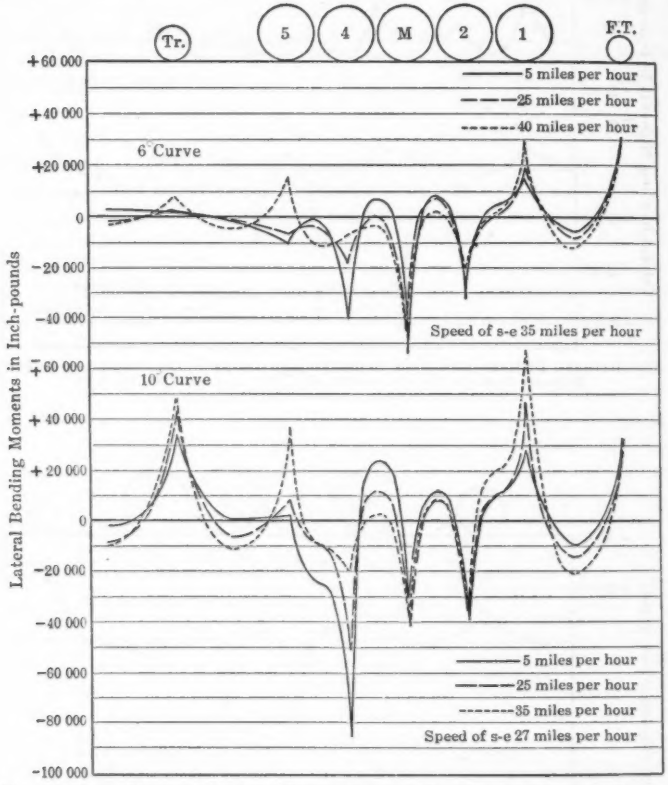


FIG. 85.—COMBINED LATERAL BENDING MOMENTS IN THE TWO RAILS OF CURVED TRACK WITH THE HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

the fourth driver of this locomotive (taking the average value found) is at least four times the average value found on straight track and at least twice the highest value that may be expected.

17.—Measurements of the Lateral Deflection of the Rail and the Distortion of the Alignment of Curves.—In order to learn to what extent the rail deflects laterally and the alignment of the curve changes under the load of a locomotive and to see the conditions under which lateral bending moments of

considerable magnitude are developed, some measurements were made on both 6° and 10° curves. An instrument was devised with which the middle ordinate of a 6-ft. chord along the rail was measured. The two bearing ends of the instrument were placed in punch marks in the side of the head of the rail and the plunger of an Ames dial at the middle of the instrument was also placed in contact with the side of the head of the rail. The measurements were taken at every 3 ft. of length of rail on both inner and outer rail, thus overlapping one another. A set of readings (zero reading) with no load on the track was taken; the locomotive was then run on and a second set of readings taken, check readings also being generally made. The tests were not as complete as was desired; they show however, the nature of the movement of the rails. More tests of this nature should give information of value.

Fig. 86 shows the alignment of the track with the locomotive standing on the 6° and 10° curves, at Ribera, with reference to the original alignment. The values recorded on the diagram represent the distance the rail is deflected from its original position, the measurement being given in inches. As the measurements were taken on the head of the rail a part of the distortion of the alignment may be due to tilting of the rail on the tie. Necessarily, in enlarging the small dimensions sufficiently to show the new position of the rails, the lines on the diagram appear distorted and also the axles are not shown as parallel to each other. Attention is called to the position of the rail joints; their smaller lateral stiffness evidently contributes to the lateral deflection.

It is seen that with the Mountain type locomotive on the 10° curve the inner rail deflects more than ½ in. from its original position and that the gauge of the track is increased from 4 ft. 8⅞ in. to as much as 4 ft. 9½ in. The outward deflection of the outer rail at the front truck wheel and first driver is large, 0.35 in. at the first driver. The greatest deflection is that of the inner rail under the third driver, 0.54 in. The changes from positive to negative bending between wheel points are evident.

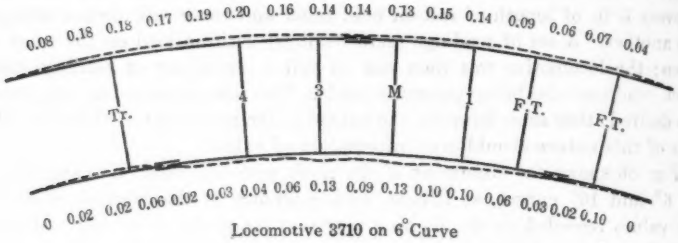
With the Mountain type locomotive on the 6° curve, the lateral deflection of both rails is much less, that of the inner rail under the third driver being only 0.13 in., and the gauge of the track was increased only about ¼ in.

With the Double Trailer Santa Fe type locomotive on the 10° curve, the inner rail under the fourth driver was deflected 0.7 in. The outer rail under the fourth driver was deflected inwardly of the track 0.14 in., so the gauge of the track was increased 0.56 in. There was also an outward deflection of the outer rail of 0.42 in. at the first trailing wheel.

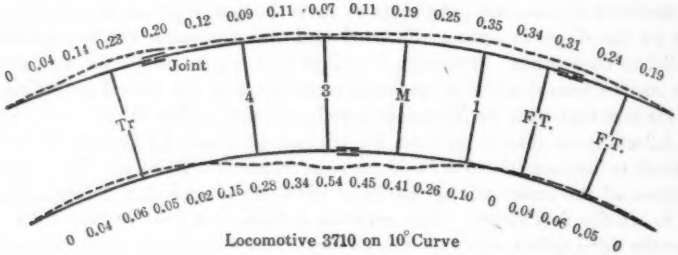
The general shape of the alignment which the rails take and the position of the many points of contraflexure are in general accord with the lateral bending moment diagrams given in Article 16. It is plain that the alignment of a curve changes materially as the locomotive passes around it and that the track as a whole has large and varied stresses put into it.

18.—Effect of Speed and Counterbalance on Curves.—It has already been found that the average of the stresses in the two rails of curved track under all the wheels of a locomotive, at a speed of 5 miles per hour, found by dividing the sum of the stresses under all the wheels of the locomotive for both rails by the number of wheels, in general agrees closely with the average of the corre-

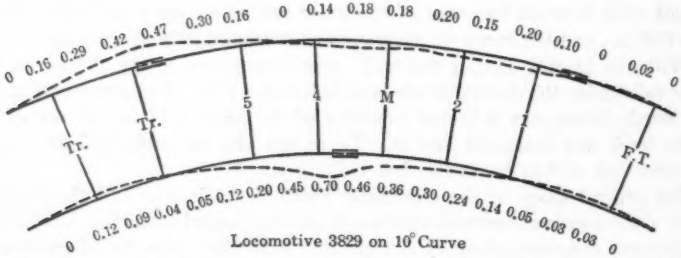
sponding stresses under the same wheels on straight track. If the average stress under all the wheels for the two rails of any curve is found for each of the speeds of any test and compared with the corresponding average stress on straight track, it will be seen that the average stress on the curved track is generally no higher and sometimes lower than that on straight track, or, at



Locomotive 3710 on 6° Curve



Locomotive 3710 on 10° Curve



Locomotive 3829 on 10° Curve

FIG. 86.—CHANGES IN THE ALIGNMENT OF THE RAILS OF CURVED TRACK WITH THE LOCOMOTIVE STANDING ON THE CURVE.

any rate, that the effect of speed on the average stress under the locomotive as a whole does not differ greatly on curves from that found on straight track, even though the transfer of weight from wheel to wheel on one side of the locomotive and from the wheel at one end of an axle to that at the other, found on curves, does change the stresses under individual wheels very greatly. The

lateral bending stresses in rails on curves, of course, are not involved in the foregoing statement.

The effect of counterbalance on the stresses in rail on curved track is much more variable than that found on straight track. In a few cases well defined counterbalance curves could be drawn through the plotted points, but generally no line seemed representative of the effect of counterbalance. The points are scattered throughout a large vertical range, and there is much more variation in the results than is found on straight track. In estimating the stress that will include the effect of counterbalance, the best that can be done is to take the average stress throughout the revolution of the driver and then allow for a very much greater variation in the stress above and below this line than would be done for straight track. One interesting result of the study of the data is that, under a number of the wheels, the stresses at the outside edge of the rail showed marked effect of counterbalance, whereas those at the inside edge show almost no counterbalance effect.

The effect of speed on the division of the load of the locomotive between the two rails of the curve is quite another matter. In most cases, the total load on the outer rail transmitted by all the wheels of one side of the locomotive bears the numerical relation to the total load on the inner rail that would be given by the analytical calculation of the effect of transverse inclination of track and centrifugal force at the given speed. This means that at slow speeds the load on the inner rail is considerably greater than the normal load, the vertical bending stresses in the inner rail with the Mikado type locomotive running at a speed of 5 miles per hour being found to be as much as 37% greater with the super-elevation of 8.5 in. on the 6° curve of the Delaware, Lackawanna and Western Railroad than on straight track. At and near the speed of super-elevation, in most of the tests on curved track, the vertical bending stresses in the two rails do not differ much from each other. It may be noted that analysis shows that the speed may be increased considerably beyond the speed of super-elevation before the load on the outer rail would become equal to that imposed on the inner rail at low speeds. Although the ratio varies with the degree of curve and the super-elevation used, it may be said that for the track used in the tests the speed may be increased 30 to 40% beyond that of super-elevation before the load on the outer rail may be expected to equal that found on the inner rail at low speeds. The foregoing discussion refers to the sum of the loads under all the wheels; the distribution of load among the several drivers may be quite unequal, the vertical bending stresses in the rail under one driver of one type of locomotive being as much as double what might be expected in the case of the 10° curve and 50% more on the 6° curve.

The lateral bending stresses developed in the rails also vary with the speed. The lateral bending stresses in the inner rails are very great at the slow speeds, especially at the driver in front of the rear driver, and sometimes the one in front of this, in the case of the locomotives having eight and ten drivers. For medium speeds, say, up to two-thirds of the speed of super-elevation, the lateral bending stress under the rear intermediate driver continues to be of considerable magnitude. For the higher speeds, the lateral bending stresses under

these wheels decrease markedly and at the speed of super-elevation the highest lateral bending stress in the inner rail may not differ greatly from the highest stress found in the outer rail. The greatest lateral bending stress in the outer rail under a driver is found at the outer front driver in those locomotives in which the wheel of the leading truck does not assume the principal part in changing the direction of the locomotive. With an increase in speed, this lateral bending stress increases. As the stresses in the outer rail under the wheels of the front truck do not increase with an increase in speed, it is evident that the greater effort to change the direction of the locomotive at the higher speeds is taken by the flange of this first outer driver.

The effect of speed on the stress at the two edges of the base of rail (combined vertical bending stress and lateral bending stress) is well shown in Figs. 64, 65, 66, and 67. A marked decrease in the stress at the outside edge of the base of the inner rail under the driver in front of the rear driver with increase of speed is very noticeable. The increase of stress in both edges of the outer rail under the companion driver is also apparent. It may be noted also that the stresses at the four edges of the two rails under the wheels of the tender generally approach uniformity of magnitude at the speed of super-elevation.

19.—Effect of Degree of Curve and Super-Elevation.—It has been seen that both the vertical bending stress and the lateral bending stress have wide ranges in value in the two rails and for the individual wheels of the several locomotives used. It may be expected that these differences will vary with the degree of curve and the super-elevation of the outer rail, although it may not be possible to distinguish between the effects of the two. In comparing the action of the locomotives, it will be convenient to refer to the values given in Figs. 69 to 72 and Figs. 75 to 78.

With the Santa Fe type locomotive on the 10° curve, at 5 miles per hour, the vertical bending stress of 29 200 lb. per sq. in. in the inner rail under the fourth driver may be contrasted with the stress of 19 400 lb. per sq. in. on the 6° curve having the same super-elevation, 4.7 in., the corresponding stress on straight track being about 12 000 lb. per sq. in. It is evident that vertical bending stress under this driver, which must be considered an excessive stress for both curves, increases with the increase in curvature, although the curves have the same super-elevation. At the speed corresponding to the super-elevation, the stress under this wheel in both curves decreased nearly to what might be expected under an even division of load among the drivers. The vertical bending stress in the inner rail of the 10° curve under the first driver, at a speed of 5 miles per hour, is less than would be expected, and decreases with an increase of speed, while on the 6° curve it remains nearly constant for the three speeds. The vertical bending stress in the outer rail of the 10° curve under the first driver, at 5 miles per hour, is not much higher than that which would normally be expected. The high value of 27 000 lb. per sq. in., at 35 miles per hour, is, of course, at a speed well above that of super-elevation. The vertical bending stresses in the outer rail under the third and fourth drivers are small; that under the fourth driver, at 5 miles per hour, is very small, showing that the load transmitted by this driver is only a small proportion of its normal load; the driver opposite it on the inner rail takes most of the load. Even at the

highest speeds, the stresses in the outer rail of both curves under this driver are below normal values. It is evident that on both curves the distribution of the load among the drivers given by the equalizing system is markedly different from that on straight track. The lateral bending stress in the inner rail, at 5 miles per hour, is very great under the fourth driver, 23 500 lb. per sq. in. on the 10° curve and 11 800 lb. per sq. in. on the 6° curve, which are very severe stresses in both cases. Even at 25 miles per hour, the stress recorded for the 10° curve was 15 600 lb. per sq. in. At the highest speed, the lateral bending stresses in the inner rail become much smaller. The lateral bending stresses in the outer rail under the first driver, indicative of the participation of this driver in changing the direction of the locomotive, at a speed of 5 miles per hour, are 12 600 lb. per sq. in. on the 10° curve and 8 800 lb. per sq. in. on the 6° curve. These lateral bending stresses increase markedly from the low speed to the highest, becoming 22 600 lb. per sq. in. on the 10° curve, at 35 miles per hour, and 13 200 lb. per sq. in. on the 6° curve, at 40 miles per hour. Bearing in mind that the lateral bending stresses at the outer wheel of the front truck vary little with changes in speed, the results indicate that altogether a larger amount of lateral pressure is required to change the direction of the locomotive at the higher speeds than at the low speeds. The amount of the lateral pressure given by the front driver also increases somewhat with the degree of curve, although the influence of the curvature does not seem to be as great as that of speed.

With the Mountain type locomotive on the 10° curve, at 5 miles per hour, the greatest vertical bending stress in the inner rail (average stress) is under the third driver, 26 700 lb. per sq. in. On the 6° curve, the vertical bending stresses are high under the first, second, and third drivers, that under the third driver, 19 700 lb. per sq. in., being about the same as that under the other two. An increase of speed to that corresponding to the super-elevation brings the vertical bending stresses in the inner rail of two curves under the fourth drivers down to about the same average value as the others, although that under the third driver on the 10° curve, 20 600 lb. per sq. in., remains greater than the others. At speeds above the speed of super-elevation, the vertical bending stresses in the inner rail have decreased materially and are much the same on both curves for all the drivers. The vertical bending stresses in the outer rail, particularly under the second and third drivers, are somewhat smaller on the 10° curve than on the 6° curve, these wheels being on the same axle as those giving very high vertical bending stresses in the inner rail. The stresses normally to be expected are not found until a speed higher than that of super-elevation is reached. The lateral bending stress in the inner rail is greatest under the third driver at 5 miles per hour, 19 800 lb. per sq. in. on the 10° curve and 11 800 lb. per sq. in. on the 6° curve. These values are roughly proportional to the degree of curve. On the 6° curve, a small value is found at the speed of super-elevation, but on the 10° curve, even at 25 miles per hour, the lateral bending stress is 14 000 lb. per sq. in., although the stress decreases rapidly with further increases of speed. In the outer rail, due probably to this locomotive having a four-wheel leading truck, the first

driver does not participate in changing the direction of the locomotive, except at the higher speeds. At 50 miles per hour, on the 6° curve, the outward lateral bending stress under the first driver becomes 7 800 lb. per sq. in. It is worth noting also that the outward bending stress in the outer rail under the fourth driver, on the 6° curve, reaches 11 800 lb. per sq. in., at a speed of 50 miles per hour. At the lower speeds, it is evident that there is a transfer of load from the outer to the inner rail on both curves, but the change is the greater on the 10° curve. It is apparent that the lateral bending stress in the inner rail for low speeds is a function of the curvature, the super-elevation being the same.

With the Mikado type locomotive on the Delaware, Lackawanna and Western Railroad, the even distribution of vertical bending stresses in the inner rail under all the drivers, at a speed of 5 miles per hour, is very apparent for all three curves, but the stresses differ on the three curves, and it may be a question as to what extent these differences in stress are due to super-elevation and to what extent they are due to curvature. The vertical bending stresses under the four drivers average about 13 000 lb. per sq. in., on the 4° curve (3.7 in. super-elevation, 105-lb. rail), 18 000 lb. per sq. in., on the 7½° curve (6.4 in. super-elevation, 105-lb. rail), and 22 400 lb. per sq. in., on the 6° curve (8.5 in. super-elevation, 92-lb. frictionless rail). It may be noted that this uniformity of stress under all the drivers is quite different from that found with the locomotives on the track of the Atchison, Topeka and Santa Fe Railway. The vertical bending stresses in the outer rail have small values under all the drivers, particularly under the main driver. They do not reach the values found in the inner rail until a speed well above that corresponding to super-elevation has been reached. The lateral bending stresses in the inner rail are much more marked in the 6° curve and are somewhat higher in the 4° curve than in the 7½° curve. It is not clear that the super-elevation is the cause of this variation. It is worth noting that, although this locomotive has a two-wheel leading truck, the lateral bending stresses in the outer rail under the first driver, at a speed of 5 miles per hour, show that this driver does not participate in changing the direction of the locomotive, except on the 4° curve. On the 6° curve, which has the frictionless rail, the lateral bending moments in the outer rail at the first driver are negative for all speeds.

With the Pacific type locomotive of the Atchison, Topeka and Santa Fe Railway, on the 10° curve at Fort Madison, Iowa, the super-elevation being only 2 in., the vertical bending stress in the inner rail under the three drivers, at a speed of 5 miles per hour, is nearly double that in the outer rail. This goes to show that in this case the curvature and not the super-elevation is the main cause of shifting the load of the locomotive to the inner rail. With the Pacific type locomotive of the Delaware, Lackawanna and Western Railroad, the shift of load to the inner rail is somewhat greater on the 7½° curve than on the 4° curve; both the curvature and the super-elevation may be responsible for this. Considering the difference in the rail sections, the lateral bending stress in the inner rail, at 5 miles per hour, is really greater on the

7½° curve than that found on the 10° curve of the Atchison, Topeka and Santa Fe Railway; it is lower still on the 4° curve. The outer rail shows no turning action by the front driver on any of the three curves, at a speed of 5 miles per hour, most of the change of direction evidently being accomplished by the wheels of the front truck. At the higher speeds, the front driver participates in the turning action in the case of the 4° curve and the 10° curve, but not the 7½° curve.

To sum up the foregoing: On all the curves and for locomotives having eight and ten drivers, at the lower speeds the vertical bending stress is markedly greater in the inner rail under the next to the last driver than that in either rail under any other driver; for locomotives of the Pacific type the stress in the inner rail is greater, but to a somewhat less degree, under all three drivers than that in the outer rail. The vertical bending stress in the outer rail is correspondingly small under the companion driver of the one giving the high stresses in the inner rail. These effects are especially noticeable at the low speeds; they continue at moderate speeds, and generally disappear at or above the speed of super-elevation. Although part of the increased value of the vertical bending stress in the inner rail is due to the transfer of load to the lower rail by reason of the transverse inclination of the track, the excess over what may be expected on straight track is greater than may be accounted for by the super-elevation, and, frequently, this discrepancy is unexpectedly great. It appears then that the equalizing system acts differently on curves than on straight track and that much more load is concentrated on one inner driver, the outer driver being relieved of a corresponding amount of load. This phenomenon was found on all the curves. Although this lack of even distribution of stresses under the drivers of one side was not present in the tests with the Pacific type locomotive, yet the vertical bending stresses in the inner rail with this locomotive are higher than may be explained by the transverse inclination of the track. It will be seen that on all the curves an increase of speed to one at or above the speed corresponding to the super-elevation results in vertical bending stresses under the different wheels of the two rails more nearly what may be expected when the effects of transverse inclination and centrifugal force are considered.

The lateral bending stress in the inner rail under the driver giving the large vertical bending stresses and also under the one ahead of it is quite marked at the lower speeds on all the curves. Although the value of the stress is considerably greater on the 10° curve, its magnitude is of considerable importance on the 6° and 4° curves.

The lateral bending stress in the outer rail under the first driver, resulting from the flange of this driver assisting in the change of direction of the locomotive, when found at the low speed, seems not to increase much with increased curvature of the track. As has already been noted, this lateral bending stress does increase with an increase in speed.

20.—Results of Tests on 10° Curves in California.—The data of tests made by the Atchison, Topeka and Santa Fe Railway and the Southern Pacific Company, in April, 1922, on the tracks of the two railways, as they relate

to the purposes of this report, have been made available to the Committee. These tests were conducted on a 10° curve of the Southern Pacific Company, at Bealville, Calif., and on a 10° curve of the Atchison, Topeka and Santa Fe Railway at Cajon, Calif. The locomotives were the Heavy Santa Fe type of the Atchison, Topeka and Santa Fe Railway and the Heavy Santa Fe type of the Southern Pacific Company, both being equipped with oil burners. The rail at Bealville was 90-lb. A. R. A.-A. section; that at Cajon was 90-lb. S. F. section. As already noted, the properties of these rails are nearly identical. At Bealville, there was 5 in. of new rock ballast on a thick mixed rock ballast foundation. At Cajon, there was 10 in. of gravel ballast. The former gave the stiffer track.

The purpose of the tests was to determine the action and rigidity of the locomotives used in the tests. On the tests made at Cajon, a flangeless tire was substituted for the flanged wheel ordinarily used for the fourth driver, the third driver being flangeless as usual. As used at the first part of the tests, the bearing part of the tread of the fourth driver was cylindrical. Later, a groove to approximate the conditions after wear was machined into the tread. Speeds of 10 and 18 miles per hour were used at both locations. The runs were made on 2.2% compensated grade, net grades of 1.8% on the Atchison, Topeka and Santa Fe Railway and of 2.0% on the Southern Pacific Company. Runs were made in two directions—up grade and down grade. The up-grade tests were made with the locomotive working steam and pulling from 9 to 12 loaded cars, the load pulled being altogether approximately 500 tons. The draw-bar pull, as measured by the dynamometer car placed immediately behind the tender, recorded about 30 000 lb. on the various runs. On the down-grade tests, the locomotives ran light and without working steam, that is, coasting.

Figs. 87 to 94, inclusive, show the average stresses for the several runs at the inside edge and outside edge of both inner and outer rails of the 10° curves at Bealville and Cajon, and at the two speeds. Figs. 95 and 96 give the values of the vertical bending stress and the lateral bending stress at Bealville and Figs. 97 and 98 the values at Cajon. Attention should be called to the fact that, in the down-grade tests at Bealville, with both locomotives, it was raining and misting all day and the rails were continually wet; this seems to have had an effect on the stresses developed under one of the locomotives and to a less extent on the other.

The stresses in the rails with the locomotive of the Atchison, Topeka and Santa Fe Railway, at Bealville, are, in general, very much the same in magnitude and character as those found with a locomotive of the same class on the 10° curve at Ribera, N. Mex., except that, on account of the locomotive used in California being equipped with oil burners, the weight on the trailers was only about three-fourths of that of the locomotive used in New Mexico, and the stresses under the trailers were correspondingly smaller. The stresses with the locomotive of the Southern Pacific Company differed in some important respects from those under that of the Atchison, Topeka and Santa Fe Railway, as did those with the locomotive with modified drivers used at Cajon.

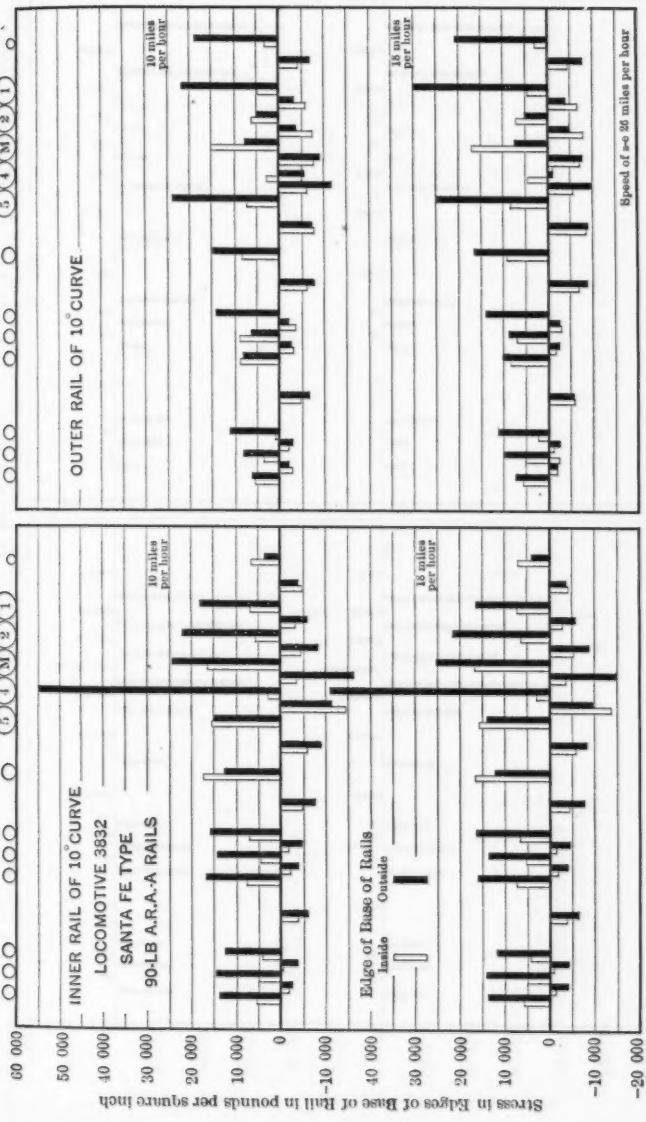


FIG. 87.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER AND OUTER RAILS OF THE 10° CURVE AT BEALVILLE, CALIF., SERIES 5500–5503, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY, PULLING 540 TONS UP-GRADE.

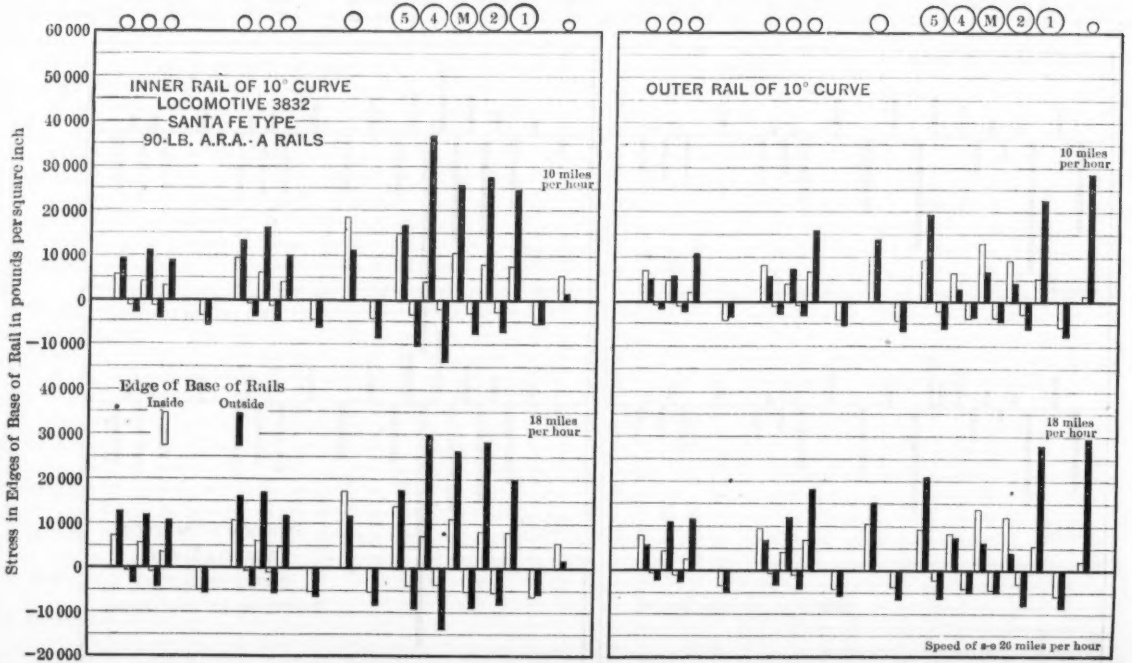


FIG. 88.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER AND OUTER RAILS OF THE 10° CURVE AT BEALVILLE, CALIF., SERIES 5504–5507, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY. DOWN-GRADE, LIGHT.

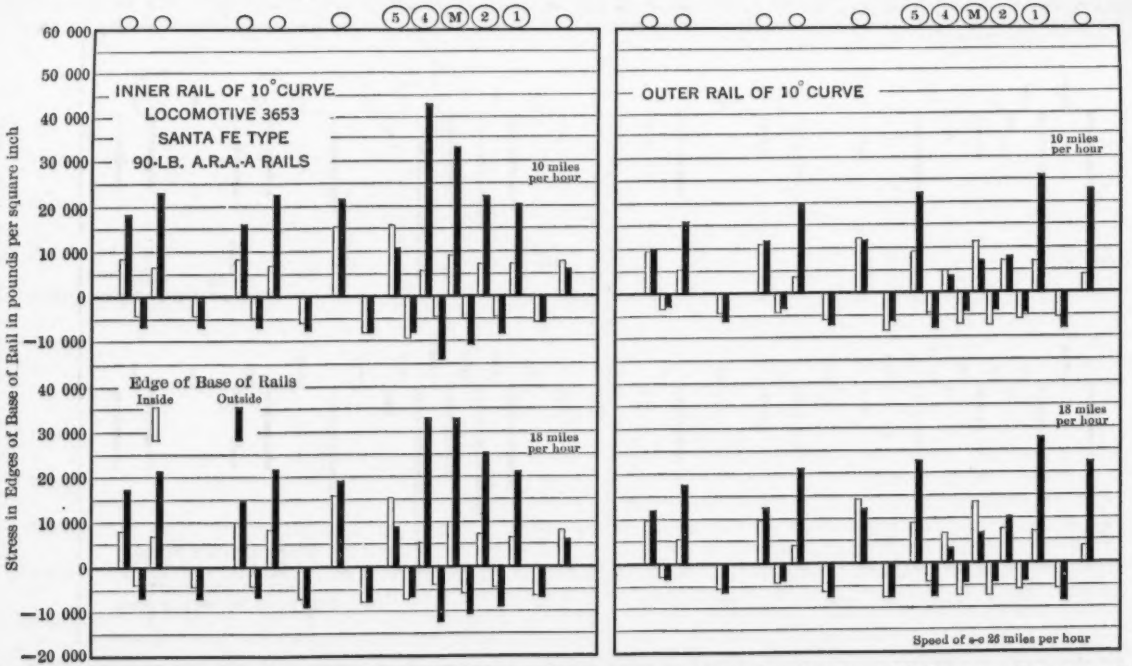


FIG. 89.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER AND OUTER RAILS OF THE 10° CURVE AT BEALVILLE, CALIF., SERIES 5513–5516, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE SOUTHERN PACIFIC COMPANY, PULLING 540 TONS UP-GRADE.

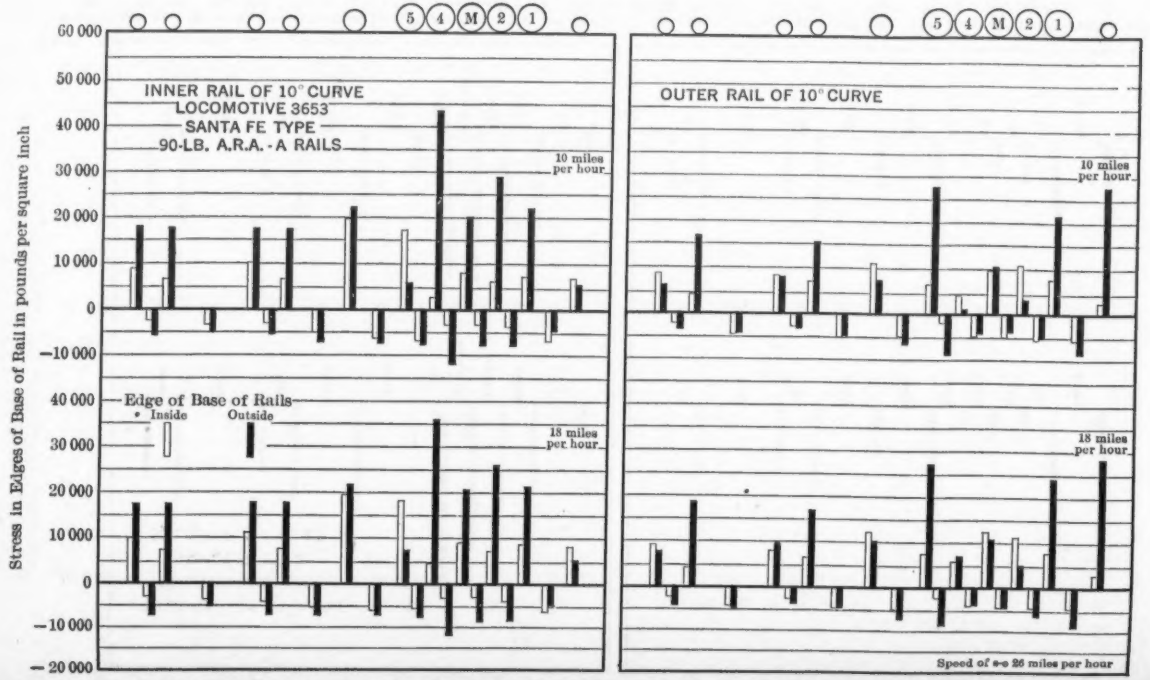


FIG. 90.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER AND OUTER RAIL OF THE 10° CURVE AT BEALVILLE, CALIF., SERIES 5508–5512, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE SOUTHERN PACIFIC COMPANY. DOWN-GRADE, LIGHT.

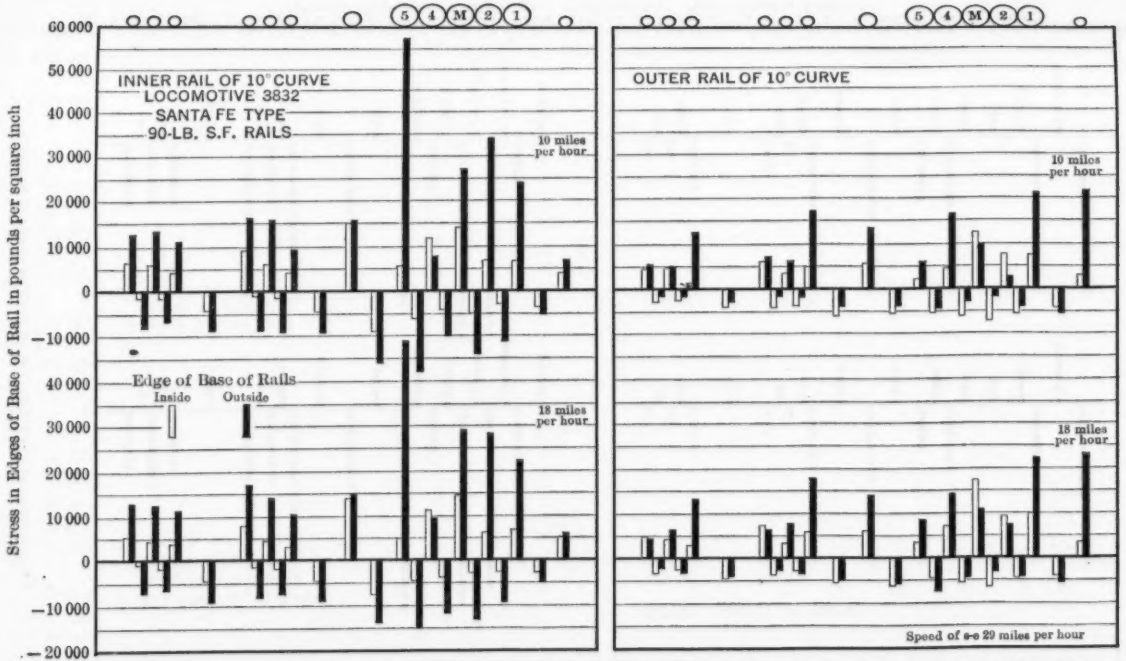


FIG. 91.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER AND OUTER RAILS OF THE 10° CURVE AT CAJON, CALIF., SERIES 5523–5526, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY, WITH FOURTH DRIVER FLANGELESS AND TREAD CYLINDRICAL. PULLING 500 TONS UP-GRADE. DOWN-GRADE, LIGHT.

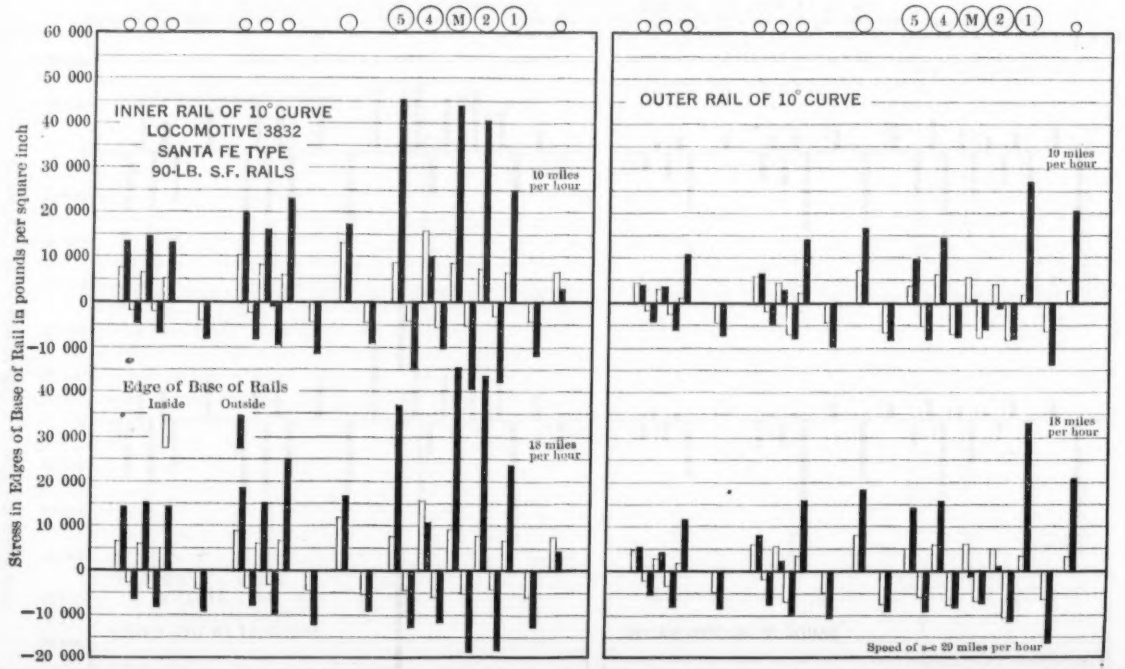


FIG. 92.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER AND OUTER RAILS OF THE 10° CURVE AT CAJON, CALIF., SERIES 5519–5522, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY, WITH FOURTH DRIVER FLANGELESS AND TREAD CYLINDRICAL. DOWN-GRADE, LIGHT.

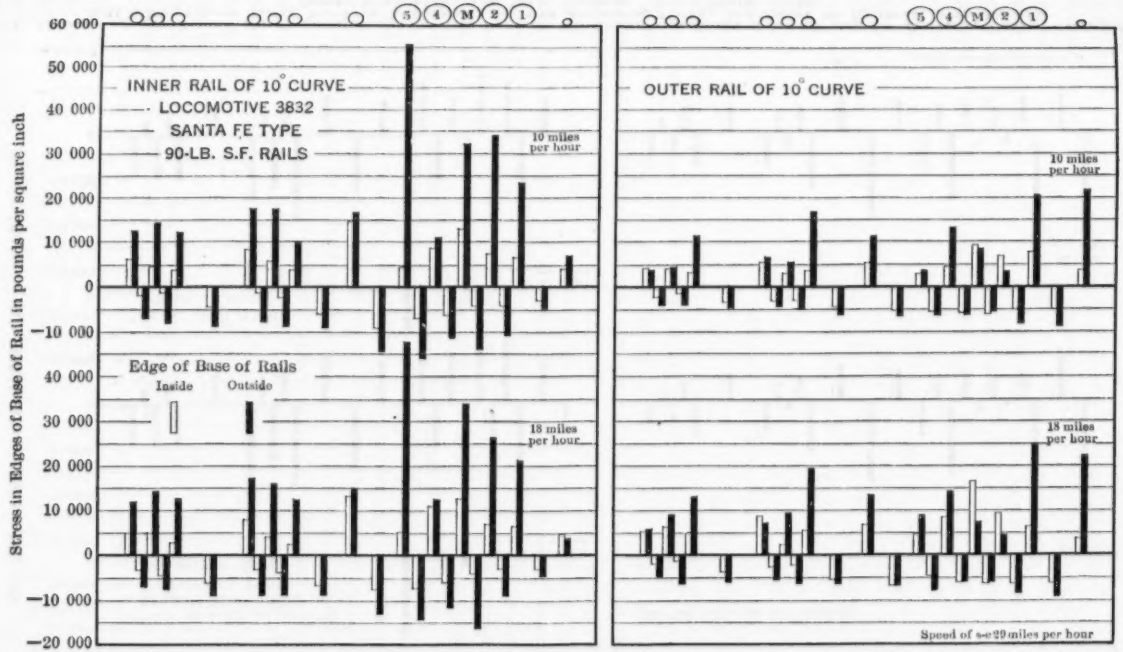


FIG. 93.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER AND OUTER RAILS OF THE 10° CURVE AT CAJON, CALIF., SERIES 5527–5530, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY, WITH FOURTH DRIVER FLANGELESS AND TREAD GROOVED. PULLING 500 TONS UP-GRADE.

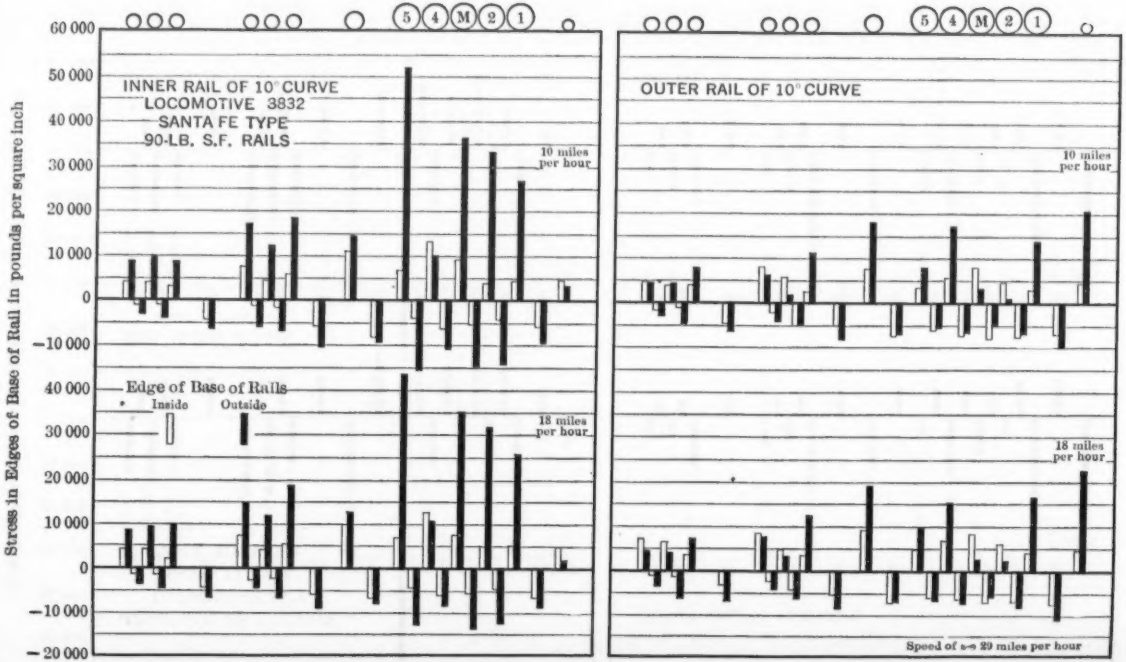


FIG. 94.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER AND OUTER RAILS OF THE 10° CURVE AT CAJON, CALIF., SERIES 5531–5534, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY, WITH FOURTH DRIVER FLANGELESS AND TREAD GROOVED. DOWN-GRADE, LIGHT.

Table 23 shows that loads at individual wheels calculated from the vertical bending stresses, at a speed of 10 miles per hour, are excessively high, namely, 50 000, 52 000, and 60 000 lb. These high loads do not necessarily occur under the same drivers, the variations seemingly being dependent on the presence or absence of flanges. It will be seen, from Fig. 95 and Fig. 97, that the vertical bending stresses under the inner driver that gives the high vertical bending stress and, therefore, transmits the high load, are nearly as great at 18 miles per hour as at 10 miles per hour, the speed of super-elevation being 26 miles per hour.

Figs. 99 and 100 give the average stress for the several runs at the inside and outside edges of both inner and outer rail of the 10° curve at Bealville when the locomotives were backing down grade at 10 miles per hour. The rails were dry. The records of the locomotives of the two companies are given.

The results of the tests on the 10° curves in California will be discussed with those of the other tests in Articles 21, 22, and 23, on "Type of Locomotive", "Effect of Pulling and Coasting and of Condition of Rail", and "Tests with Locomotive Backing over Curved Track", respectively.

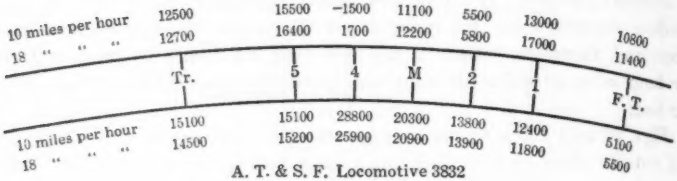
21.—Type of Locomotive.—On straight track the stresses developed in the rail are controlled by such matters as the wheel loads and the wheel spacing. Lateral bending stresses are developed by reason of the coning of the wheels, the variations in the movement of the locomotive, and by irregularities in track conditions. Variations in the design of locomotives, such as the length of wheel-base, the number of drivers, the use of two-wheel or four-wheel trucks, may be made without bringing unduly high stresses into the rail, provided proper care is used in making the design. On curved track, other considerations will be found to enter into the development of stresses in the track. The change of direction of the locomotive in going around a curve is effected by pressure between the flanges of the outer wheels of the front truck and the outer rail and frequently, also, by the outer first driver, resulting in a lateral bending of the outer rail. An unexpected effect is the greatly increased load transmitted to the inner rail by one or more of the drivers at low and medium speeds and a corresponding decrease in the load on the opposite driver, the conditions producing excessive vertical bending stresses in the inner rail. The lateral spreading action on the inner rail develops marked lateral bending stresses in that rail. In all these matters, the characteristics of the type of locomotive will be found to influence the action of the locomotive and the nature and amount of the stresses developed in the rail under the various wheels.

In the matter of changing the direction of the locomotive, it seems evident from the tests that the four-wheel truck of the Pacific and Mountain types presents advantages over the two-wheel truck of the Santa Fe and Mikado types. With the four-wheel leading truck used, the loads on the wheels are such as to give moderate vertical bending stresses. The lateral bending stresses in the outer rail under the outer wheels of the four-wheel truck on curves from 4° to 10°, may be termed moderate. With the locomotives having

VERTICAL BENDING STRESSES

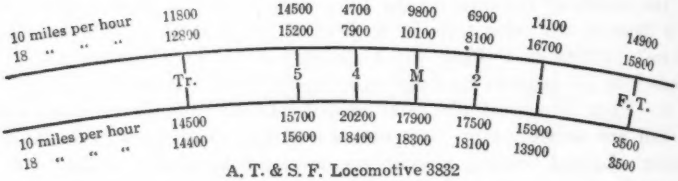
10° Curve, Bealville, California

Superelevation 4.4 in. for 26 miles per hour



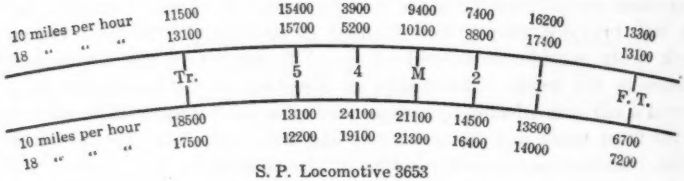
A. T. & S. F. Locomotive 3832

Pulling 540 Tons Up-grade



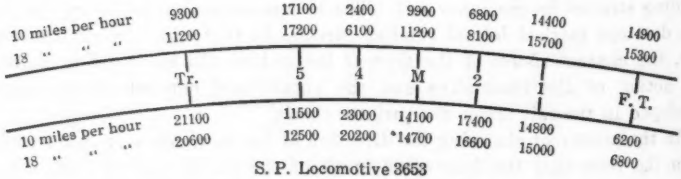
A. T. & S. F. Locomotive 3832

Down-grade, Light, – Wet Rails



S. P. Locomotive 3653

Pulling 540 Tons Up-grade



S. P. Locomotive 3653

Down-grade, Light, – Wet Rails

FIG. 95.—VERTICAL BENDING STRESES IN BASE OF RAILS OF THE 10° CURVE AT BEALVILLE, CALIF., HEAVY SANTA FE TYPE LOCOMOTIVES OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY AND THE SOUTHERN PACIFIC COMPANY.

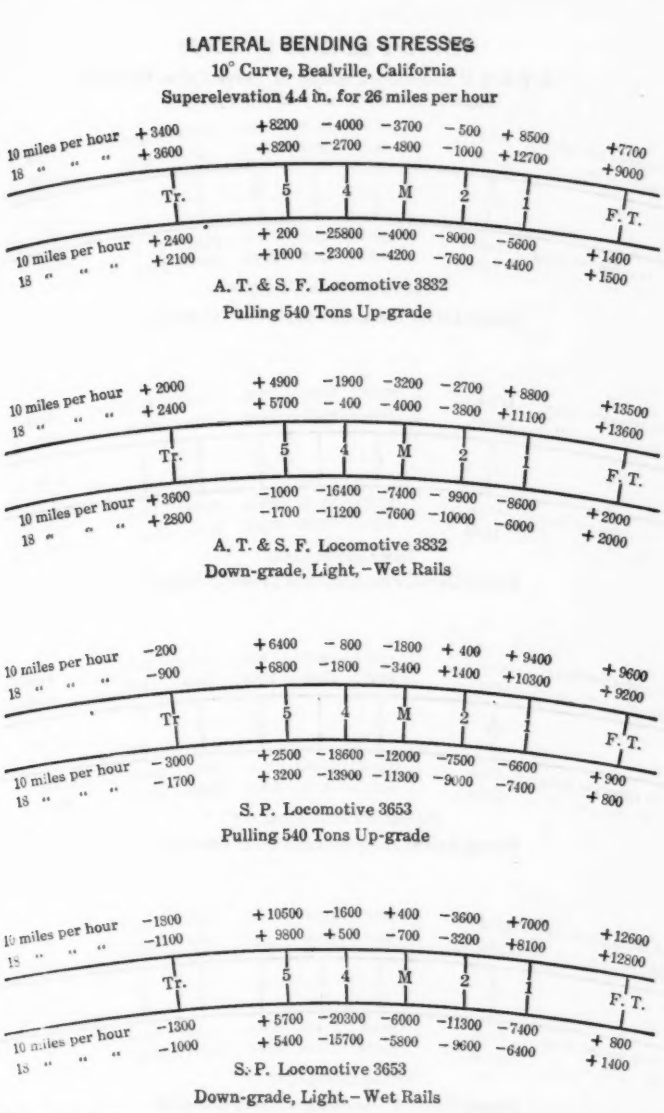
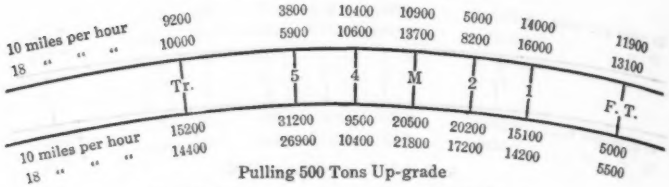


FIG. 96.—LATERAL BENDING STRESSES IN BASE OF RAILS OF THE 10° CURVE AT BEALVILLE, CALIF., HEAVY SANTA FE TYPE LOCOMOTIVES OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY AND THE SOUTHERN PACIFIC COMPANY.

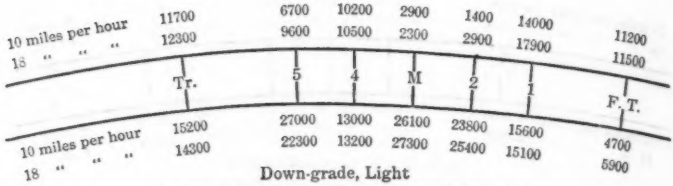
VERTICAL BENDING STRESSES

A. T. & S. F. Locomotive 3832 on 10° Curve, Cajon, California
Superelevation 5.4 in. for 29 miles per hour



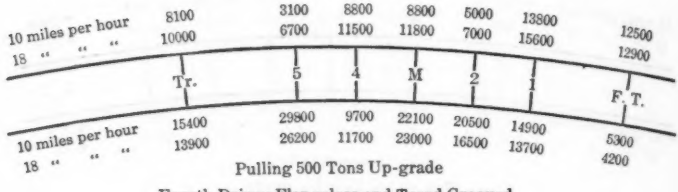
Pulling 500 Tons Up-grade

Fourth Driver Flangeless and Tread Cylindrical



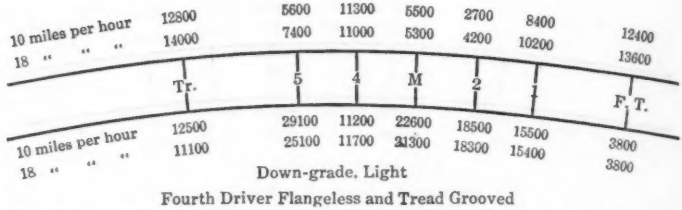
Down-grade, Light

Fourth Driver Flangeless and Tread Cylindrical



Pulling 500 Tons Up-grade

Fourth Driver Flangeless and Tread Grooved



Down-grade, Light

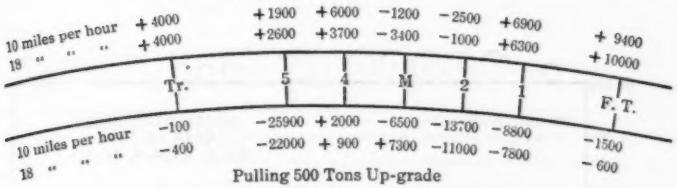
Fourth Driver Flangeless and Tread Grooved

FIG. 97.—VERTICAL BENDING STRESSES IN BASE OF RAILS OF THE 10° CURVE AT CAJON, CALIF., HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

LATERAL BENDING STRESSES

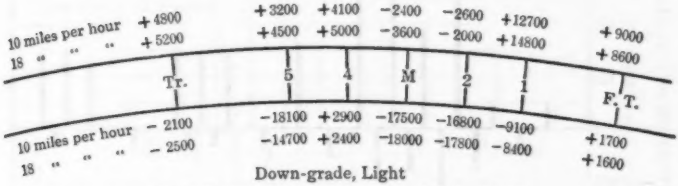
A. T. & S. F. Locomotive 3832 on 10° Curve, Cajon, California

Superelevation 5.4 in. for 29 miles per hour



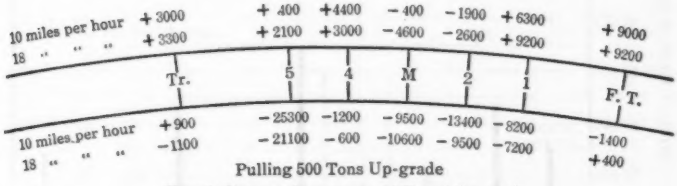
Pulling 500 Tons Up-grade

Fourth Driver Flangeless and Tread Cylindrical



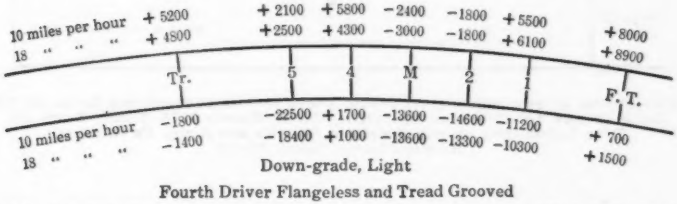
Down-grade, Light

Fourth Driver Flangeless and Tread Cylindrical



Pulling 500 Tons Up-grade

Fourth Driver Flangeless and Tread Grooved



Down-grade, Light

Fourth Driver Flangeless and Tread Grooved

FIG. 98.—LATERAL BENDING STRESSES IN BASE OF RAILS OF THE 10° CURVE AT CAJON, CALIF., HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY.

the four-wheel trucks, there is little or no outward bending stress developed in the outer rail under the first driver, this driver participating in the turning action only at the highest speeds. With the two-wheel front truck, the lateral bending stress in the outer rail under the first driver may become

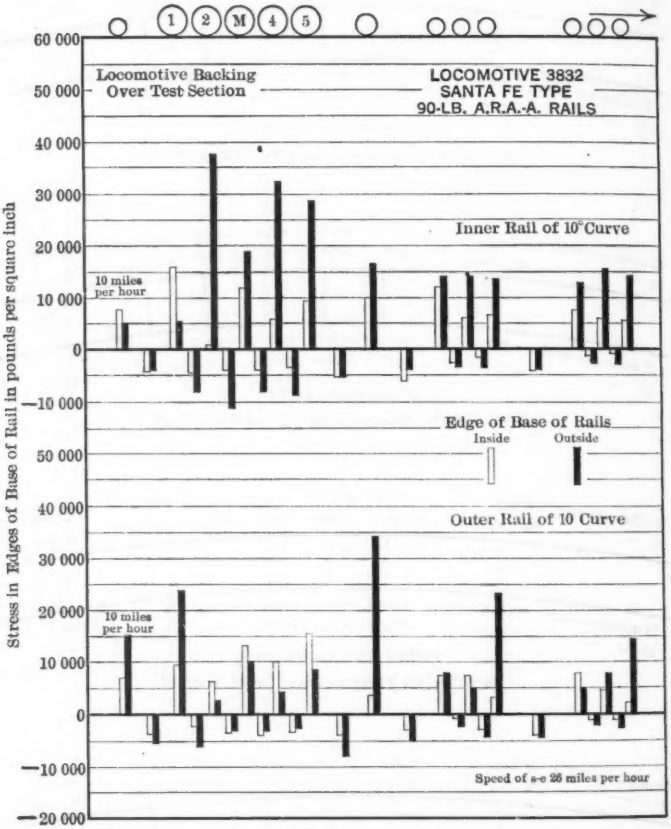


FIG. 99.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER AND OUTER RAILS OF THE 10° CURVE AT BEALVILLE, CALIF., SERIES 5518, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE ATCHISON, TOPEKA AND SANTA FE RAILWAY, BACKING DOWN GRADE, LIGHT.

very great, and the value, 22 600 lb. per sq. in., found under the first driver of the Santa Fe type on the 10° curve, at a speed of 35 miles per hour, may be considered a very excessive stress. It would appear that the use of a four-wheel truck would improve the conditions under the front outer driver of

this locomotive. In the case of the Double Trailer Santa Fe type, the load has evidently been redistributed in such way as to give an increased vertical load on the wheels of the front truck. (It will be recalled that this locomotive was not re-designed for the use of the four-wheel trailer.) The resulting

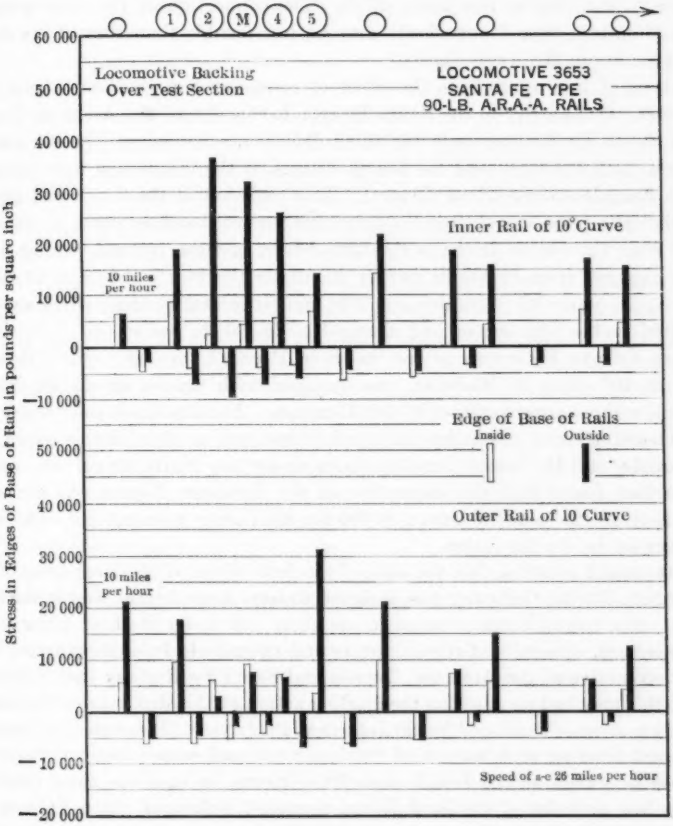


FIG. 100.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER AND OUTER RAILS OF THE 10° CURVE AT BEALVILLE, CALIF., SERIES 5517, HEAVY SANTA FE TYPE LOCOMOTIVE OF THE SOUTHERN PACIFIC COMPANY, BACKING DOWN GRADE, LIGHT.

lateral bending stress at the outer wheel of the truck is very great, 16 000 lb. per sq. in.

The lateral bending stress in the inner rail under the intermediate drivers of these locomotives is so important that its presence and possible magnitude

are matters that should receive careful consideration in designing such locomotives. It is evident that the magnitude of the lateral bending stress in the inner rail may be expected to depend on the wheel base, although the lateral bending stress under the third driver of the Mountain type on the 10° curve is nearly as great as that under the fourth driver of the Santa Fe type, 19 800 lb., as compared with 23 500 lb., per sq. in. The driver wheel base of the former is 18 ft. and that of the latter, 22 ft. The connection of the front truck of the Mountain type (four-wheel) may be less flexible than that of the Santa Fe type locomotive (two-wheel).

It is of interest to note the effect of removing flanges from intermediate drivers. Ordinarily, on the Santa Fe type locomotive of the Atchison, Topeka and Santa Fe Railway only the third drivers are flangeless. In the tests at Cajon, both the third and the fourth drivers of the locomotive were equipped with flangeless tires (blind drivers). This omission of the flange and the use of the cylindrical tire shifted the very high vertical bending stress in the inner rail from the fourth driver to the fifth driver, and the vertical bending stress was changed from 29 000 lb. per sq. in. under the fourth driver to 29 000 lb. per sq. in. under the fifth driver. The highest lateral bending stress was also shifted to the fifth driver and changed to 25 900 lb. per sq. in. The Heavy Santa Fe type locomotive of the Southern Pacific Company, used in the tests on the 10° curve at Bealville, was equipped with flanges on all the drivers, which is the regular practice of this Company. In these tests, the greatest lateral bending stress was under the fourth driver, but the third driver helped considerably, and the lateral bending stress under the fourth driver was smaller than that found with the locomotive of the Atchison, Topeka and Santa Fe Railway on the same 10° curve, 18 600 for the former as compared with 25 800 lb. per sq. in. for the latter.

It should be noted that the axle of the first driver of the locomotive of the Southern Pacific Company was designed to have some lateral play in the bearings. The lateral bending stress in the outer rail under the first driver at the speeds used, differed so little in nature and magnitude from those found with the locomotive of the Atchison, Topeka and Santa Fe Railway that it appears that this play had no effect on the guiding action of this driver or on the lateral bending stresses developed by the intermediate drivers. It seems, also, that the outward bending or deflection of the inner rail and consequent temporary increase of gauge at the fourth and fifth drivers, in case the third driver is flangeless, and also at the third driver, in case it is flanged, throw light on the ability of a rigid wheel-base to pass around a sharp curve without permanently spreading the track, even though the flange of the outer rear driver keeps away from the outer rail.

The lateral bending stresses under the trailers of the Mountain and Santa Fe types have high values on the 10° curve, although they are moderate on the 6° curve. It is evident that there is sufficient freedom in turning to give low lateral bending stresses on the 6° curve, but that the restraint on the 10° curve develops high lateral pressures against the outer rail. The double trailer Santa Fe type, however, has such freedom of motion that relatively small lat-

eral bending stresses are developed under these wheels on the 10° curve. It should also be recorded that the lateral springs regulating the lateral swing of the trailer truck in the Mountain type locomotive were loosened for one day's runs; the lateral bending stresses developed in the two rails of the 10° curve with the trailer truck in this condition did not differ particularly from those found when the springs were in their usual condition.

22.—Effect of Pulling and Coasting and of Condition of Rail.—In the discussion of the tests on straight track, it has been shown that the stress in rail under the wheels of the locomotive does not differ greatly whether the locomotive is pulling a load or coasting. On the curved track at Bealville, the average of the vertical bending stresses in base of rail for both outer and inner rails of the 10° curve at all the wheels of the locomotive did not differ noticeably whether the locomotive was pulling a load of 500 tons up grade or whether it was coasting down grade. This load was about one-half the full capacity of the locomotive in operating on the 2.2% grade. The stresses under individual wheels, however, showed some differences.

At Cajon, the stress in the outside edge of the inner rail under the fifth driver (see Figs. 91 to 94) was somewhat greater when the locomotive was pulling load than when coasting for those tests when the tire of the fourth driver was cylindrical (typical of new blind tire), and only slightly greater in the tests, when the fourth driver was grooved. It is seen then that the lateral bending stresses were about the same in the two cases. The vertical bending stresses in the outer rail under the fifth driver were somewhat smaller when the locomotive was pulling. The stresses in the inner rail under the second and third drivers were somewhat smaller when the locomotive was pulling a load than when coasting. The difference in the stresses in the outer rail at the first driver and at the wheel of the front truck (and also the lateral bending stress at other wheels) does not indicate a change that can be considered characteristic. Altogether no very marked differences in stress under the various wheels are noticeable in the two ways of operating the locomotive.

The comparison of the results of the tests at Bealville is complicated by the fact that the down-grade tests were made on wet rails, while in the up-grade tests the rails were dry. It is seen, however, that the vertical bending stress under the rear driver, which is the location giving apparently somewhat higher stresses when the locomotive is pulling load and the one for which higher stresses may possibly be expected for this condition if at all, does not differ in either rail for the up-grade and down-grade tests with either locomotive. It is apparent, however, that there is a considerable difference in the distribution of the stresses under the intermediate drivers due, no doubt, to differences in friction of the wheel on the rail for the two conditions of wet and dry rail and, therefore, to differences in the position of the center of rotation in the change of direction of the locomotive on the curve. At the outside edge of the inner rail, under the fourth driver of the locomotive of the Atchison, Topeka and Santa Fe Railway, the stress drops from 54 700 lb. per sq. in. when the rail is dry (up grade, pulling) to 36 600 lb. per sq. in. when the rail is wet (down grade, light). The stresses under the first, second, and third drivers are some-

what less with the dry rail. The lateral bending stresses in the outer rail under the first driver and the wheel of the front truck are higher with the wet rail. With the locomotive of the Southern Pacific Company, a reduction in stress with the wet rail is found at the main driver, that at the fourth driver remaining the same. It is evident that the condition of the rail has an effect on the development of stress at the different wheels.

23.—Tests with Locomotive Backing over Curved Track.—On curved track, with the locomotive moving forward, the guiding and the change of direction around the curve are effected by the flanges of the outer wheel of the front truck and generally, also, by that of the first driver. When the direction of the locomotive is reversed, it is to be expected that the turning action will be effected by the fifth driver and also by the trailer if its connections are not too flexible. It is also to be expected that the inner second driver will cause large lateral and vertical bending stresses in the rail at the point where the fourth driver develops these large stresses when the locomotive is moving forward. That this reasoning is correct is shown by an examination of Figs. 99 and 100, which give stresses in rail when the Santa Fe type locomotives of the two railroads were backed around the 10° curve at 10 miles per hour, the rails being dry in these tests. The magnitude of the stress at these wheels is not as great as may be expected. It will be noted that, with the locomotive of the Atchison, Topeka and Santa Fe Railway, the turning action is effected by the pressure of the flange of the trailer against the outer rail, while with the locomotive of the Southern Pacific Company the turning action is taken more largely by the fifth driver. Evidently, the trailer of the former locomotive is more restrained in its connections. This restraint was also noticed with a locomotive of the same type in the tests on the 10° curve at Ribera (the locomotive moving forward), but as that restraint did not appear on the 6° curve at Ribera, it would seem that there is sufficient freedom of action in the connection of the trailer not to develop lateral bending stresses under the trailer on the 6° curve.

24.—Lateral Bending of the Rail Section.—When a lateral force is applied to the head of the rail and the base of the rail is held in place either wholly or partly, it may be expected that the rail web will bend and the head of the rail will deflect outwardly or inwardly relatively to the original vertical axis of the rail. To determine whether this lateral bending of the rail section is a measurable quantity, an instrument was made, which was fastened firmly to the base of the rail and had a vertical standard rising at one side of the rail. An Ames dial at the level of the middle of the depth of the head of the rail was attached to this standard; its plunger bore against the side of the head of the rail. The locomotive was run by at a slow speed, 1 to 2 miles per hour, and readings were taken as a wheel passed the instrument and again between wheels. The observation proved to be definite and positive, and succeeding runs confirmed the readings. It should be noted that the observations are independent of any tilting of the rail about its base or its support on the tie; such tilting may also occur.

Measurable values of the lateral bending of the rail section were found. Reference will be made only to the general nature of the results. The deflec-

tion varied from zero to 0.016 in. The values on the 10° curve were somewhat higher than on the 6° curve. The Santa Fe type gave somewhat higher values than the Mountain type. As would be expected from the low speed used, the deflections were greatest in the inner rail, and the direction of deflection in the inner rail was generally away from the track. The direction of the deflection in the outer rail did not always correspond with the character of the observed lateral stress in the base of rail. As the lateral bending of the rail itself is the result of a number of forces, including those acting at the supports of the rail, it is possible that the sign of the bending moment at a wheel point may not be in keeping with the direction of the lateral force acting at that point. An interesting feature occurred when the Santa Fe type was backed over the track. A very high value of the deflection was observed in the inner rail under the fifth driver, a deflection of 0.016 in. away from the track for the 10° curve, and one of 0.014 in. for the 6° curve. The absence of leading wheels to guide the driving wheel-base is evident in these results. Values of the deflection as great as 0.007 in. were found at the wheels of the tender, the middle wheel of a truck on the inner rail giving large deflections.

25.—Preliminary Tests on the Illinois Central Railroad.—Preliminary to the main tests on curved track heretofore described, tests were made on two curves on the Illinois Central Railroad for the purpose of developing the method of testing curved track, of learning the technique necessary for a successful test, and of trying out the instrumental equipment. The results are of sufficient interest to warrant recording.

Fig. 101 gives the stresses in the inner and the outer rail of a 7° curve at Murphysboro, Ill., with a Mikado type of locomotive (No. 1861) of the same general class as that used in the tests on straight track, described in the second progress report, the rail being 90-lb. A. R. A.-A. section, whereas in the earlier tests on straight track the rail was 85-lb. Am. Soc. C. E. section. The super-elevation of the track was $3\frac{3}{4}$ in., corresponding to a speed of 28 miles per hour. It is seen that the turning action is taken by the flanges of the outer front truck wheel and the first driver, and there is little difference in lateral bending stresses under these wheels at the two speeds used. The fourth driver and the trailer push outwardly on the outer rail at both speeds; the trailer also acts to increase the curvature of the inner rail. The main lateral bending stresses in the inner rail are at the second and third drivers. The vertical bending stress at the third driver, at a speed of 5 miles per hour, is 80% more than that on straight track and, at 25 miles per hour, it is 30% greater than on straight track. In the outer rail, the stresses at the second and third drivers are very small.

Fig. 102 gives the stresses in the inner and outer rail of a 14° curve connecting the Havana Branch at Champaign, Ill. The locomotive is a ten-wheel type (4-6-0). The wheel loads are relatively small. The rail is 85-lb. section. The super-elevation of the track, $3\frac{3}{4}$ in., corresponds to a speed of 20 miles per hour. It is seen from the stresses that the turning of the locomotive at both speeds is produced by the action of the flanges of the two outer wheels of the front truck. The third driver pushes outwardly on the outer rail at both speeds. The first and second drivers push outwardly on the inner rail, the

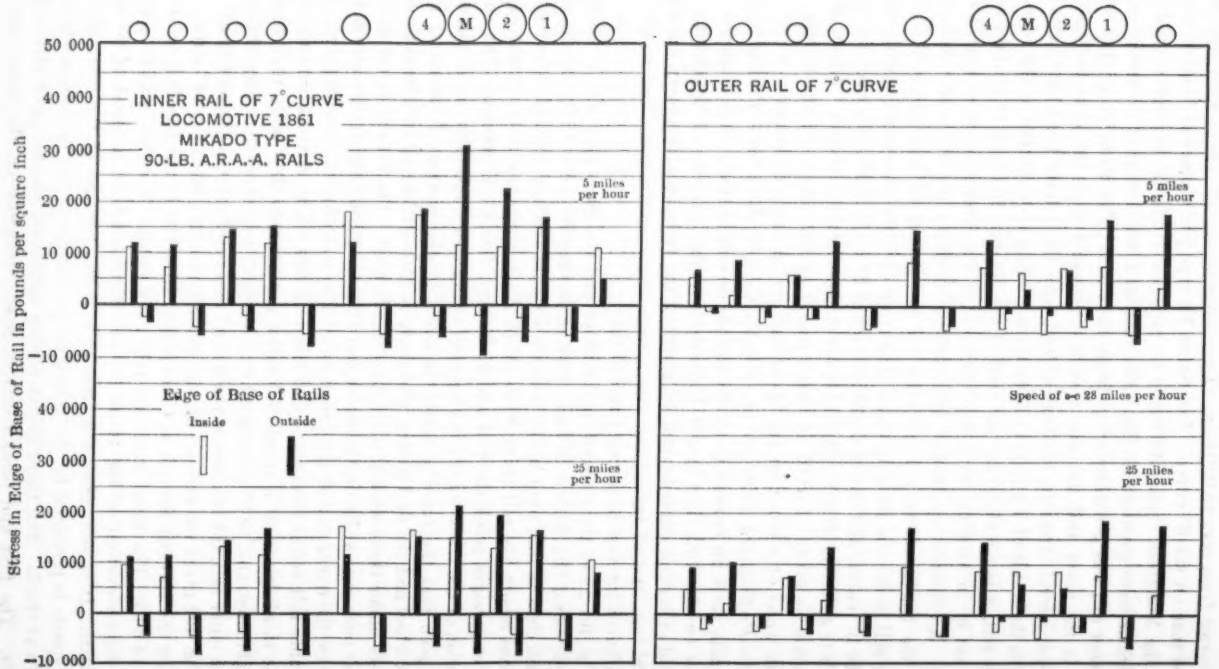


FIG. 101.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF INNER AND OUTER RAILS OF THE 7° CURVE, SERIES 5056–5062, MIKADO TYPE LOCOMOTIVE OF THE ILLINOIS CENTRAL RAILROAD.

TYPE LOCOMOTIVE OF THE ILLINOIS CENTRAL RAILROAD.

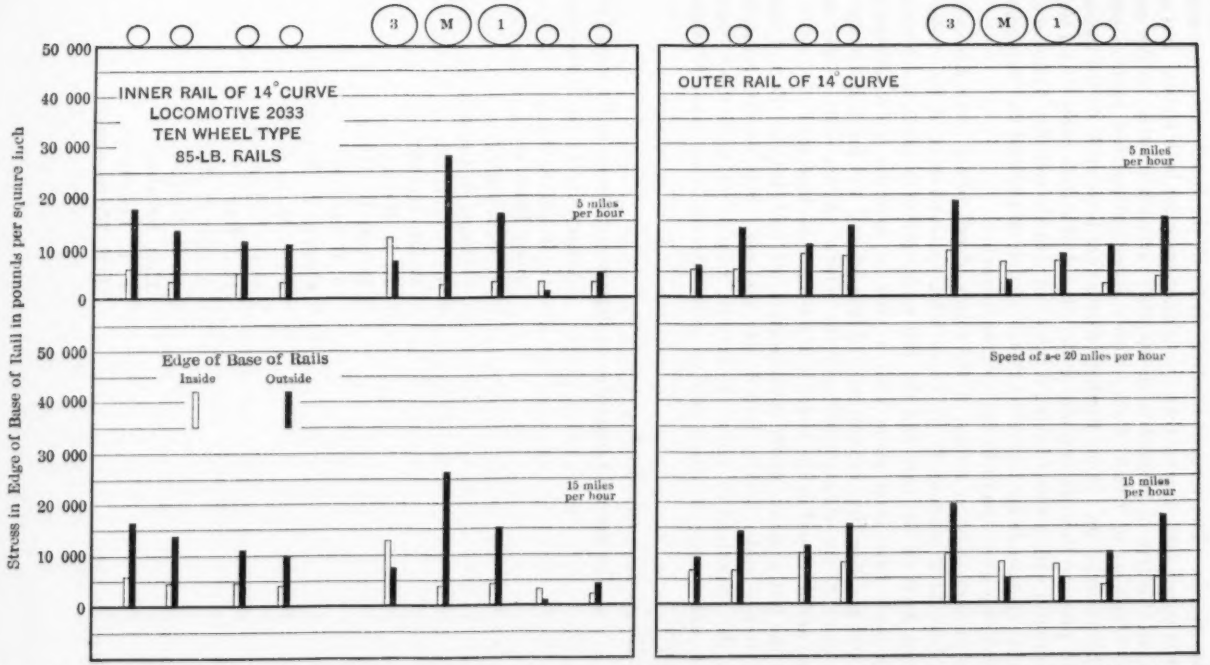


FIG. 102.—STRESS AT THE INSIDE AND OUTSIDE EDGES OF THE BASE OF THE INNER AND OUTER RAILS OF THE 14° CURVE, SERIES 5035–5041, TEN-WHEEL TYPE LOCOMOTIVE OF THE ILLINOIS CENTRAL RAILROAD.

lateral bending stress under the second driver being of considerable magnitude at both speeds. The vertical bending stress in the inner rail at the second driver is nearly 50% greater than on straight track for both speeds. It is seen that even this type of locomotive shows great variation in stress on this curve.

26.—Progress of Other Work.—Other investigations not reported herein have been carried on. Some further progress has been made on the transmission of pressure through granular materials. A few tests made with a loaded car equipped with a wheel having a large flat spot showed that strains of considerable magnitude are caused by this condition and proved the practicability of measuring these strains by the stremmatograph.

Laboratory tests on several of the forms of rail joints commonly used in track have been conducted. In these tests, measurements of the strains in different parts of the bars and the rail under ordinary loads and somewhat higher loads have been made with a view of learning where stresses are developed and how they are transmitted between rail and joint, in an effort to learn more of the characteristics of the joints. A few tests to find what stresses are developed in the joints in track have also been made.

In the test program under consideration are included the effect of canted rails, stresses under heavily loaded cars, and tests with electric locomotives where the conditions of center of gravity, relation of unsprung to sprung weight, and spacing of wheels may be quite different from those of the steam locomotive.

Respectfully submitted,

The Special Committee to Report on Stresses in Railroad Track,

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JOHN BRUNNER,
W. J. BURTON,
CHARLES S. CHURCHILL,
W. C. CUSHING,
W. M. DAWLEY,
H. E. HALE,
ROBERT W. HUNT,
J. B. JENKINS,
GEORGE W. KITTREDGE,
PAUL M. LA BACH,
C. G. E. LARSSON,
G. J. RAY,
ALBERT F. REICHMANN,
H. R. SAFFORD,
EARL STIMSON,
F. E. TURNEAURE,
J. E. WILLOUGHBY.

November 29, 1922.

DISCUSSION

S. TIMOSHENKO,* ESQ. (by letter).—The problem of dynamic stress in rails is one of great complexity, and only by systematic experiments is it possible to separate the effects of counterbalance, flat spots on wheels, low spots on rails, and various other causes of dynamic stress. The Special Committee on Stresses in Railroad Track, in its second† and third‡ progress reports, studies separately and in detail the effect of counterbalance. The other causes are not brought out, and their average effect is marked under "speed effect." Although the average value of this speed effect is not "unreasonably high", to use the words of the report, the observed values of stress that are recorded, show, for the several wheels and rail sections, considerable variations for which no explanation is given.

In this discussion, it will be shown that, in some cases, this variation can be attributed to the vertical motion of the wheel due to the causes mentioned previously. By taking into consideration the yielding of the track acting as a spring, it is easily shown that the effect of the counterweights is greater than that given by static formulas, and explanation is obtained for the fact that the maximum stress occurs somewhat after the counterweight has reached its lowest position.

The additional stresses produced in the rail by the vertical motion of the wheel, due to a flat spot or other irregularities in the wheel or rail, are readily calculable. It is shown that the additional deflection produced by a flat spot is proportional to the depth of the spot and depends on the time required for the wheel to pass over the flat spot. From this, it follows that the more rigid rail gives a somewhat higher proportional increase of this stress with increase of speed than a less rigid one.

The method of calculating additional stresses due to vertical motion of a wheel will be shown in the very simple case of a single wheel. Considering the track as an elastic structure, the following relation between the pressure, P, and the deflection, y, of the rail can be obtained:§

$$P = k\,y \qquad (a)$$

in which,

$$k = \sqrt[4]{64\,E\,I\,u^3} \qquad (b)$$

$E\,I$ denotes the flexural rigidity of the rail, and u denotes modulus of elasticity of rail support defined as the pressure per unit of length of each rail required to depress the track one unit.

Considering the rail as a spring, the rigidity of which is determined by Equation (b), the equation of free vibration of the wheel on the rail can be written in the well known form:

* Research Dept., Westinghouse Elec. & Mfg. Co., East Pittsburgh, Pa.
† *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1191.
‡ See p. 907.
§ See First Progress Report, *Bulletin*, Am. Ry. Eng. Assoc., 1918. p. 887.

$$\frac{q}{g}\frac{d^2y}{dt^2} + ky = 0 \qquad (c)$$

in which, q denotes the weight of the wheel, together with the half weight of the axle and the weights of other parts rigidly connected to the axle.*

The period of free vibration of the wheel on the rail, from Equation (c), will be,

$$\tau = 2\pi\sqrt{\frac{\lambda}{g}} \qquad (d)$$

in which,

$$\lambda = \frac{q}{k} \qquad (e)$$

denotes the static deflection of the rail under the pressure, q.

Assuming, for instance, for a 100-lb. rail, that $EI = 30 \times 10^6 \times 44$ lb. in.^{2}, and $u = 1\,500$ lb. per inch length of rail per inch depression, and $q = 3\,000$ lb., Equation (d) gives $\tau = 0.0483$ sec., that is, the wheel performs about 20 vibrations per sec.

Consider now different forces acting on the wheel: The forces acting on the wheel through the spring may be considered as constant† and the corresponding depression of the rail may be calculated from Equation (a).

The vertical component of the centrifugal force, C, due to the unbalanced rotating parts, will be,

$$C\cos\left(\frac{2\pi t}{\tau_1} - \alpha\right)$$

in which τ_1 denotes the period of one complete revolution of the wheel and α is a constant depending on the initial position of the counterweight.

The vertical motion of the wheel produced by this force may be found from the well known equation of forced vibrations:

$$\frac{q}{g}\frac{d^2y}{dt^2} + ky = C\cos\left(\frac{2\pi t}{\tau_1} - \alpha\right) \qquad (f)$$

By integrating Equation (f), the expression for the forced vibration may be obtained in the form:

$$y = \frac{C}{k}\cdot\frac{1}{1 - \left(\frac{\tau}{\tau_1}\right)^2}\cos\left(\frac{2\pi t}{\tau_1} - \alpha\right) \qquad (g)$$

in which $\frac{C}{k}$ is the deflection, obtainable from the static Equation (a). It is seen that the amplitude of the forced vibration, Equation (g), is greater than $\frac{C}{k}$ in the ratio,

* The mass of the vibrating part of the rail is negligible in comparison with the mass of the wheel.

† The variation of this force, due to the vibration of the locomotive on its springs, can also be taken into consideration without difficulty, as the period of this vibration may be considered to be large, in comparison with τ and the static formulas may be used in calculating the deflection of the rail.

$$1 : \left[1 - \left(\frac{\tau}{\tau_1}\right)^2\right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (h)$$

When the time, τ_1, of a revolution of the wheel is large as compared with the period, τ, of free vibrations of the wheel, the ratio, Equation (h), differs little from unity and the static Equation (a) may be used without essential errors. Assuming, for instance, $\frac{\tau}{\tau_1} = \frac{1}{4}$ that is, taking about 5 rev. of the wheel per second:

$$1 : \left[1 - \left(\frac{\tau}{\tau_1}\right)^2\right] = 1.067$$

The maximum deflection, therefore, due to the counterbalance, is about 7% greater than the value given by the static Equation (a). The ratio, Equation (h), increases with the diminishing of the time, τ_1, of a revolution. If, for instance, the effect of the inertia forces of the second order, is taken, for which the period is equal to $\frac{1}{2}\tau_1$ we obtain, for the example just given,

$$1 : \left[1 - \left(\frac{\tau}{\frac{1}{2}\tau_1}\right)^2\right] = 1.33$$

that is, the difference between the results obtainable from Equation (a) and Equation (g) is about 33 per cent.

As C is proportional to the square of the speed, and the ratio, Equation (h), increases with the speed, it may be concluded from Equation (g) that the stress due to counterbalance increases much further than the square of the speed.

By taking into consideration the damping effect of friction, it is easy to show that the maximum depression, due to the counterbalance, must occur somewhat after the counterweight has reached its lowest position.

Consider, now, the effect of a flat spot on the deflection of the rail. Let η denote the variable depth of the flat spot and y the deflection of the rail. Then, the vertical displacement of the wheel is equal to $y + \eta$, and the differential equation of motion of the wheel in the vertical direction, due to the effect of the flat spot only, will be,

$$\frac{q}{g}\frac{d^2(y+\eta)}{d\,t^2} + k\,y = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (i)$$

from which,

$$\frac{q}{g}\frac{d^2 y}{d\,t^2} + k\,y = -\frac{q}{g}\frac{d^2\eta}{d\,t^2} \ldots\ldots\ldots\ldots\ldots\ldots (j)$$

When the shape of the flat spot and the speed of the locomotive are known, η can be easily expressed as a function of the time. Suppose,

$$-\frac{q}{g}\frac{d^2\eta}{d\,t^2} = \zeta\,(t)$$

in which, ζ is a known function of the time. Then, from Equation (j), we obtain,

$$\frac{q}{g}\frac{d^2 y}{d\,t^2} + k\,y = \zeta\,(t) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (k)$$

By comparing the forms of Equations (f) and (k), it is seen that the effect of the flat spot is the same as that of an external force, $\zeta\,(t)$. The deflec-

tion produced by this force at the moment, t, may be calculated by using the known formula for forced vibrations,

$$y = \frac{g}{q} \cdot \frac{\tau}{2\pi} \int_0^t \zeta(t_1) \sin \frac{2\pi}{\tau} (t - t_1)\, d\, t_1 \quad \ldots\ldots\ldots\ldots\ldots (l)$$

As an example, consider the effect of a flat spot the shape of which is determined by,

$$\eta = \frac{f}{2} \left(1 - \cos \frac{2\pi x}{l}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (m)$$

in which l denotes the circumferential length of the flat spot, and f the depth of the spot at the middle of its length. Substituting in Equation (m), $x = v\,t$, in which v is the speed, we obtain,

$$\zeta(t) = -\frac{q}{g} \frac{d^2\,\eta}{d\,t^2} = -\frac{q}{g} \cdot \frac{f}{2} \frac{4\,\pi^2\,v^2}{l^2} \cos \frac{2\,\pi\,v\,t}{l}$$

Substituting this in Equation (l) and integrating, the additional deflection produced by the flat spot has then the following expression:

$$y = \frac{f}{2} \cdot \frac{1}{1 - \frac{\tau_2^2}{\tau^2}} \left(\cos \frac{2\,\pi\,t}{\tau_2} - \cos \frac{2\,\pi\,t}{\tau}\right) \quad \ldots\ldots\ldots\ldots (n)$$

in which, as previously, τ denotes the period of free vibration of the wheel on the rail, and $\tau_2 = \frac{l}{v}$ denotes the time required for the wheel to pass over the flat spot. Equation (n) can be used for calculating additional deflection at any time between $t = 0$ and $t = \tau_2 = \frac{l}{v}$. This deflection is proportional to f and depends on the magnitude of the ratio, $\frac{\tau_2}{\tau}$. The variation of the deflection of the rail under the wheel in the interval $0 \leq t \leq \tau_2$, for different values of the ratio, $\frac{\tau_2}{\tau}$, is represented in Fig. 103.

It should be noted that, in the first moment, owing to the flat spot, the pressure on the rail diminishes, and the wheel begins to move vertically downward with acceleration. Then, the retardation of this movement begins and, with it, the increase in pressure and deflection. The values of the maximum deflections, calculated from Equation (n) for different values of the ratio, $\frac{\tau_2}{\tau}$, are given in Table 24.

TABLE 24.

$\frac{\tau_2}{\tau}$ =	2	$\frac{3}{2}$	1	$\frac{4}{5}$	$\frac{2}{3}$	$\frac{3}{5}$	$\frac{1}{2}$
$\frac{y_{max.}}{f}$ =	0.33	0.65	1.21	1.41	1.47	1.45	1.33

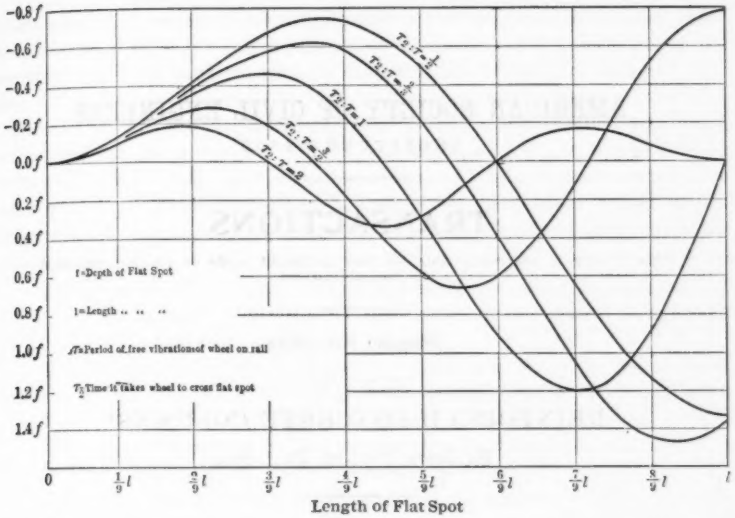


FIG. 103.

The maximum deflection, equal to about 1.47, occurs at the speed corresponding to $\frac{\tau_2}{\tau} = \frac{2}{3}$. Taking, for example, $f = \frac{1}{16}$ in., it may be calculated from Equation (a) that the force, corresponding to the additional deflection, equal to $\frac{1.47}{16}$ in., is about 12 000 lb. Thus, a comparatively small flat spot produces, at certain speeds, very appreciable dynamic effect. Similar results can be obtained for flat spots of different shapes, for instance, in the case where the variation of the depth, η, follows the parabolic law, or the law represented by a broken line.*

The vertical motion of the wheel for $t > \tau_2$ can be considered as a free vibration. The initial displacement and the initial speed of these vibrations, that is, the deflection and the speed at $t = \tau_2$, may be found from Equation (n). Owing to the damping effect, these vibrations die out very quickly.

In all the considerations presented, the mass of the vibrating part of the rail was neglected in comparison with the mass of the wheel. The error involved in this simplification of the problem is not essential if the time, τ_2, which is required for the wheel to pass over the flat spot, is long enough in comparison with the period of the fundamental type of free vibration of the rail. With increasing speed, the error increases and the elementary solution, given previously, may be considered as a first approximation. The exact solution can be obtained only on the consideration of the vertical motion of the wheel, together with the transverse vibrations of the rail.†

* See the writer's paper, "On the Stresses in Railway Track", *Transactions*, Inst. of Engrs. of Ways of Communication, Petrograd, 1915.

† Solutions of a number of problems on vibrations of rails are given in the writer's paper, "On the Vibrations of Rails", *Bulletin*, Electro-Technical Inst. Vol. XIII (1915), Petrograd. See, also, the writer's paper, "Etude de l'Action des Charges roulantes sur les Rails", *Le Genie Civil*, 1921, p. 555.

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Paper No. 1523

REINFORCED CONCRETE COLUMNS*

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With Discussion by Messrs. Edward Godfrey, E. S. Martin, George Paaswell, Jacob Feld, A. W. Buel, H. V. Spurr, Austin H. Reeves, Joel B. Cox, L. J. Mensch, John F. O'Rourke, S. H. Ingberg, H. Alden Foster, and John Tucker, Jr.

Synopsis

The purpose of this paper is primarily to determine, by scientific methods, values of safe working stresses for the several types of reinforced concrete columns, and to indicate those excessive and, therefore, unsafe stresses permitted by the 1917 and 1921 Specifications suggested by the Joint Committees on Concrete and Reinforced Concrete‡ and on Standard Specifications for Concrete and Reinforced Concrete,§ respectively, particularly the excessive and dangerous stresses permitted by the 1921 Specifications for spirally reinforced columns.‖

In Section I, the effect of the non-homogeneity of concrete on its various constants of stress and elasticity is analyzed. An analysis based on the test data and observations of all appropriate columns tested in the United States, in which the effect of the variation of the ultimate strength of the concrete is mathematically eliminated, leads to equations expressing the ultimate strength of the several types of reinforced concrete columns in terms of all the column variables. The analysis gives a clear insight into and establishes the

* Presented at the meeting of February 14, 1923.

† Pittsburgh, Pa.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1101.

§ *Proceedings*, Am. Soc. C. E., August, 1921, p. 59.

‖ The Specifications submitted by the Joint Committees on Concrete and Reinforced Concrete and on Standard Specifications for Concrete and Reinforced Concrete will be referred to in this paper as the "1917 Specifications" and the "1921 Specifications," respectively.

functioning of the several column components and the strength to be expected from them *per se*. One important result attained is the knowledge of the uniformity of strength developed by longitudinal reinforcing rods per unit area.

In Section II, the mathematical equations for the spirally reinforced concrete column are developed. By the substitution of known values for the variables of these equations, simultaneous values of longitudinal column core stress and spiral stress are obtained, that check observation values within experimental measure, thus verifying the equations. It can be shown by these equations that:

1.—The ultimate strength of spirally reinforced concrete is independent of the yield-point strength and the ultimate strength (tensile) of the spiral reinforcement.

2.—The spirally reinforced concrete column develops maximum strength and fails when the spiral is stressed at most to the yield point. The failure of the spiral in tension by further increase in column compression strain is a relatively unimportant phenomenon.

3.—It is impossible to stress the steel spiral beyond the yield point by compression on the concrete core of the column.

In Section III, the concept of reliability is analyzed. Factors governing reliability are critically examined, and a method of accurately expressing it in a reliability number determined by the well known error function is developed. The reliability number is applied to substantiate the equations of Section I and is also a necessary factor in the determination of the safe working stresses in Section IV.

In Section IV, the factors essential in the determination of safe working stresses in general are enumerated. Formulas are derived mathematically without the inclusion of the "guesswork" factor of safety or its equivalent, for the determination of safe stresses for the several types of reinforced concrete columns. Stresses calculated from these formulas for many examples within the range of possible selection of column variables are tabulated in comparison with those permitted by the 1917 and 1921 Specifications. By comparison, the stresses permitted by these specifications are shown to be inconsistent and, in many cases, unsafe.

SECTION I.—EMPIRICAL FORMULAS*

1.—Introduction

Because of the relatively large expense and the necessity of special equipment, only a small number of plain and reinforced concrete columns have been tested. In each series of tests the number of specimens, especially of duplicate specimens, has been less than desirable in order to determine definitely the effect of each variable affecting the strength of the column.

* An empirical formula is one derived directly from an experimental basis, assumed to be resolvable into simpler laws, and is only to be accepted as true for values lying within the limits of the observed quantities.

This Section is devoted to an analysis, as one large group, of the various important groups of columns that have been tested, thereby eliminating, not only the variation in results due to factors both unknown and incapable of being measured, but also the personal equation of each investigator. By thus increasing the range of variables and the number of test specimens, a surer and wider insight into the functions of the reinforcing components and a more exact derivation of empirical formulas are obtained. The formulas developed in this Section, besides being of direct benefit in the determination of the probable strength of a compound column, are applied in Section IV in the determination of safe working stresses. No common dimensions of the columns, such as length, diameter, slenderness ratio, strength of concrete, percentage of reinforcement, etc., are found in the several groups of tests. It is a difficult task, therefore, to co-ordinate all the tests and from them to determine the effect of each variable and to formulate a general law for the strength of the columns of each type. Evidently columns having concrete of different strengths and different percentages of spiral and longitudinal reinforcement, cannot be directly compared. Some means of comparison, therefore, must be found, in order to derive the benefit of including every column in the mathematical analysis. A different method of analysis must of necessity be adopted for each type of column.

2.—Importance of the Empirical Formula

The formulas in this paper are derived from an almost purely empirical basis. Such formulas are not generally held in highest esteem, but they should not, especially in this case, be considered of little importance. One important branch of mathematics, the Theory of Errors, deals with its subject-matter empirically. Scientifically, there is no such thing as an error. The "error" or discrepancy from a predicted result or from other observations is due to an accumulation of minutiæ or influences, of which knowledge is limited or lacking, or of influences too complicated to permit an attempt at a mathematical solution. Empirical formulas give no insight into underlying operating causes. Newton considered a "force" as an operating cause and a concrete thing in itself; modern science has shown that a "force" is merely a means of measuring change in motion, cannot be measured by any other unrelated means, and is incapable of being separated from matter.

In Science the attitude of endeavoring to determine the reason for phenomena has changed to one of determining the manner in which the phenomena occur; it is a change from formerly asking why to the present inquiry of how.

The engineer is most concerned with results. The empirical formula gives results accounting for the effects of all actuating influences without in any way attempting to analyze and separate the effect of each, and directly discounting the effect of the accumulation of minutiæ and unknown influences.

3.—Types of Columns

The types of columns included in this analysis are:

1.—Structural steel.

2.—Cast iron.

3.—Concrete, unreinforced (permissible up to six diameters).

4.—Concrete, reinforced:

(*a*) With longitudinal steel rods, wire laterals binding the rods at intervals about equal to the column diameter.

(*b*) With structural steel elements, these elements themselves forming a column. The steel structure as a whole, columns, floor-beams, etc., is first erected, the concrete being cast in place in forms built around the steel structure.

(*c*) With wire spiral. This type is not used in practice, building codes requiring longitudinal steel rods as reinforcement in addition to the spiral.

(*d*) With spiral and longitudinal steel rods.

(*e*) von Emperger type: A spiral and rod reinforced concrete column in the center of which is a cast-iron tube or column. This type was originated by Dr. von Emperger, of Austria.

The structural steel and cast-iron columns are included for comparison because, in most cases, they can be substituted for the reinforced concrete column and *vice versa*. The plain concrete and spiral, without rod reinforced columns are included to develop the formulas for the strength of the rod reinforced and the spiral and rod reinforced columns. Timber, stone, and brick columns, or rather piers,* not in the same category with the reinforced concrete column, are omitted.

4.—Comparison Equations

Formulas giving the reduction in strength with length for different types of columns are in common usage. These formulas, however, do not afford a means of comparing the relative reduction in strength of different types of columns. In order to afford such a comparison, Equations (1) and (2) were developed. Although a square steel column cannot strictly be compared with a round one, by substituting as the diameter, the distance from face to face of the steel column, in Equations (1) and (2), satisfactory results are obtained.

For built-up steel columns or cast-iron columns of one cross-sectional area the reduction in strength with length is not uniform for all conceivable distribution of this area. The usual variation of this distribution, however, is not great, owing to the limited possible methods of fabrication, etc. The relation between ultimate strength and length of column is shown in Equation (1), as follows:

$$U_0 = K_1 - K_2 \frac{L}{D} \qquad (1)$$

in which,

U_0 = ultimate strength of the column;

L = length of the column;

D = diameter of the column; and

K_1 and K_2 are constants.

* In *Technologic Paper No. 111*, U. S. Bureau of Standards, Mr. J. G. Bragg gives the results of a comprehensive series of tests on brick piers, by far the largest ever tested, and also a résumé of the only other important series of tests that have been made on brick piers

Equation (1) can be reduced to:

$$\frac{U_0}{K_1} = \frac{K_1}{K_1} - \frac{K_2}{K_1}\frac{L}{D}$$

or,

$$U = 1 - K\frac{L}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Equation (2) furnishes a means of comparing the relative reduction in strength with length, for various types of columns, by means of the constant, K, for each type.

5.—Unreinforced Concrete

A.—General

Concrete is exceedingly non-homogeneous, and, therefore, its properties such as strength, rigidity, etc., vary throughout its mass. Because of this non-homogeneity, the various constants, such as ultimate stress, Poisson's ratio, etc., for concrete are difficult to determine. As the constants vary when all known and controllable operating factors are made constant, not only must the average value of the constant be determined, but also the manner of its variation, that is, a number that measures a quantity corresponding to the probable error.

There is no means of accurately determining the strength, even the average strength, that concrete made from a hitherto unused aggregate will develop except by means of test specimens made in the same manner, the same proportions, percentage of water, etc., as the concrete to be used in the final structure itself.* Steel made under rigid supervision and qualifying for certain definite strength tests, is received as a finished product at the point of erection and is erected with the positive knowledge of its possessing a certain definite strength.

The actual strength of the concrete in a structure is different. Such strength cannot be determined until the concrete has been placed and test cylinders from the same mix as that of the structure have been tested.

B.—Test Specimens

Concrete test specimens are made in two standard forms: The cube, usually 6 in. along an edge, and the cylinder, two diameters in height, usually 8 by 16 in., or 6 by 12 in. The test cylinder gives directly the ultimate strength of a column made from the concrete, corrected for loss in strength due to length.

Arthur N. Talbot, Past-President, Am. Soc. C. E., states:† "The strength of columns and cylinders agrees fairly well." The strength of cylinders and columns agrees only "fairly well" owing to the natural variation in the strength of the concrete itself, irrespective of its external form. Two columns, or two cylinders, made from the same batch of concrete, will not be of the same

* Complete fundamental information on the preparation and properties of concrete is contained in *Technologic Paper No. 58*, U. S. Bureau of Standards.

† *Bulletin No. 20*, Univ. of Illinois Eng. Experiment Station.

strength, and, for the same reason, a column and a cylinder made from the same batch, except by chance, will not be of the same strength.

The cube develops considerably more strength than the cylinder, but is a form more difficult to mould properly, requires more skill to make, and gives less reliable results. From groups of auxiliary test specimens, made to determine the strength of concrete in columns,* Professor Talbot has shown experimentally the greater strength of cubes. These results show the strength of the cube to be 1.5 times that of the cylinder. The ratio of the strengths, however, must be dependent on the relative size of the test specimen and the coarse aggregate, hence there can be no fixed ratio between the strength of the concrete in the cylinder and the cube.

The greater strength of the cube as compared to that of the cylinder has a theoretical basis and is the result of the method of compression failure of concrete. Concrete in compression fails in shear in a plane or planes inclined about 35° to the axis of compression. The concrete column and the test cylinder of a length equal to twice the diameter permit this shear to occur freely. The form of the cube, however, forces the shear failure to occur in a plane inclined at an angle of 45° or more and, therefore, increases the compressive force necessary to cause this shear and concomitant failure. This method of failure must be understood for the further analysis of reinforced concrete elements.

There is always the danger that the layman or even the ordinary supervisor of tests will not appreciate the fact that the ultimate stress in the cube is much higher than that which a column of the same material is capable of developing, and, from the results of cube tests, will consider the concrete much stronger than it actually is.

The late J. B. Johnson, M. Am. Soc. C. E., states† that "the cube is not the best form" of test specimen, because of the discrepancy between column and cube ultimate stresses, but he did not explain the reason for this difference. Professor Talbot has also stated‡ that it is probable that many engineers have been misled by high values obtained on test cubes. On account of the risk just mentioned, the cube should be discarded as a test specimen for determining the merit of concrete for compression members such as columns. For highway work, etc., it is possible that it may be a better test specimen than the cylinder.

C.—Constants for Concrete

Ultimate Stress.—Concrete being a non-homogeneous material, its compressive strength varies along the compression axis of a concrete column. A column under increasing compression will fail at the weakest point in the axis, and the ultimate strength of the concrete, determined in this manner, is not its average ultimate strength, but is an accurate measure of the weakest concrete in the column. Therefore, the longer the axis, the greater the length in which failure can occur, the lower the average strength, and the less the variation in strength. This may be the only reason for the reduction in

* *Bulletin No. 20,* Univ. of Illinois Eng. Experiment Station.
† "Materials of Construction," Paragraph 113, 1919 Edition.
‡ *Bulletin No. 10,* Univ. of Illinois Eng. Experiment Station.

strength with column length. Test cylinders in compression must fail within a comparatively limited length, and for this reason alone the test cylinder, therefore, should show a higher average compressive strength than the column, but a greater variation in strength. This greater variation in strength is known; but comparatively, for the length, the test cylinder is weaker than is otherwise indicated. This lesser strength is probably due to the relative size of the largest aggregate and the diameter of the test specimen for the cylinder and the column.

Ultimate Strain.—The ultimate strain of concrete is subject to variations similar to those in the ultimate stress, and it will vary when every variable is controlled and made constant. The ultimate strain is of use only in its relationship to the ultimate stress, and is to be determined as a function of that stress, if it is one. For various specimens of one strength, it is not to be expected that the ultimate strain will also be equal or constant, nor that the ultimate strains and stresses bear an exact relationship for a group of specimens made from one batch of concrete. The strain measurements of a column or test specimen are only an average of the strains along the compression axis for the length over which these strains are measured. The ultimate strain determined in this manner is not the true ultimate, nor the strain in the proximity of the point of failure. The strain measurements on test cylinders, as in the case of those of ultimate stress, will be nearer the actual quantities. By extrapolation of the stress-strain curves for concretes of various strengths,* the three average values of ultimate strain shown by the points in Fig. 1 were obtained. The representative line for these points shows marked characteristics and an evident relationship between concrete strength and ultimate strain.

These values, obtained by extrapolation, are roughly checked from observation of the strain at which the shell of a spirally reinforced column commences to scale. Although this scaling would indicate the ultimate strain of unreinforced concrete, such concrete frequently spalls and gives indications of failure prior to ultimate stress. These indications, together with the difficulty of observing the exact stress and strain at which initial spalling occurs, make results obtained from this method less accurate than are to be desired.

Some results seem to refute the curve shown on Fig. 1, which indicates that concrete of an ultimate strength of about 600 lb. per sq. in. has an ultimate strain of 0.0006 in. per in. Apparently contradicting this are the results of Professor Talbot's tests of two concrete columns with structural steel reinforcement, the concrete having a strength of 680 lb. per sq. in. Adding this unit strength to that of the steel reinforcement to obtain the total column strength would indicate that before failure the columns were strained to at least 0.0015 in. per in. The conclusions to be drawn from these two columns of concrete of low strength are not certain or final, because some reason other than the ultimate strain of the concrete being greater than 0.0006 in. per in. may have been the cause of the strength added by the concrete. The struc-

* From the curves given in *Technologic Paper No. 12*, U. S. Bureau of Standards, *Bulletins Nos. 300 and 466*, Univ. of Wisconsin, and *Bulletin No. 20*, Univ. of Illinois, Eng. Experiment Stations.

tural steel columns restrained the concrete inside, and may have added to the strength in a manner similar to that of a spirally reinforced column. This action, however, would not be present in a column reinforced only with rods, as the rods would have no restraining action on the concrete enclosed by them.

Poisson's Ratio.—To measure Poisson's ratio for a non-homogeneous substance such as concrete is difficult. Poisson's ratio is that of the lateral to the longitudinal strain for a particle of the concrete infinitely small, when it can be assumed that the strains are homogeneous throughout the particle. Any measurements of the longitudinal or lateral strains in concrete necessarily must be averages, and it is impossible to obtain measurements of these quantities that are averages of the same set of points in the concrete. The longitudinal strain will be an average of the set of adjacent longitudinal points in the length of the column, and the lateral strain will be an average of the set of adjacent lateral points in the column. These two sets of points will have only an infinitesimal volume or only one point in common, and their ratios will be valueless for giving Poisson's ratio. A roughly approximate value of Poisson's ratio can be obtained by taking the average longitudinal strain of the column for several points around the diameter, and twelve or more measurements of the lateral strain in its length. Professors Talbot and Withey have made longitudinal and lateral measurements, but the lateral measurement was taken at only one place on the column. The computation of Poisson's ratio from these measurements leads in many cases to absurd results,

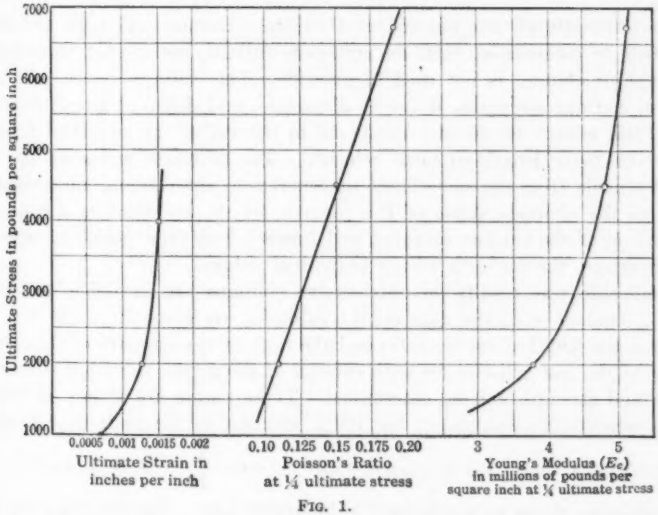


FIG. 1.

the values obtained being frequently greater than 0.5. This is impossible, being a mathematical statement regarding a substance more fluid than a perfect fluid, or a direct contradiction of the principle of the conservation of energy. Fig. 7 shows several curves giving the relation between Poisson's

ratio and stress, computed from the measurements of Professors Talbot and Withey. The results have qualitative if not quantitative value. Fig. 1 shows the relation between Poisson's ratio and the ultimate strength.

Young's Modulus.—For concrete, Young's modulus, E_c, is a variable, being a function of the stress. No part of the stress-strain curve of concrete obeys Hooke's law, that is, that E_c is a constant. The values of Young's modulus, computed from the ultimate strain and ultimate stress, are shown on Fig. 1, with the representative curve, the points representing averaged values of the quantities obtained from the same source as the values for Poisson's ratio.

Yield Point.—There is no yield point of concrete. The concept of yield point was originated for steel which has a true yield point. The proof that there is no yield point for concrete needs no other evidence than the fact that authorities cannot agree on a method of obtaining it, or arrive at the same value in their determination of it.

The properties of other materials used in a monolithic structure may make it essential that a certain strain is not exceeded, which would place a limit on the "working strain". There is a stress (as in steel), a certain definite percentage of the ultimate stress, an infinite number of repetitions of which will not cause failure, nor too large an increase in strain. These, then, are definite criteria to be observed in concrete, instead of the so-called "yield point."

6.—Unreinforced Concrete Columns

The strength of the unreinforced concrete column has been forcefully indicated by compression tests on numerous full-size specimens; nevertheless, this type of column is not used in practice. The columns tested have concrete of various strengths, different diameters, and different length-diameter ratios, and cannot be directly compared to determine the empirical formula expressing their length-strength relation. The ultimate stress of the test cylinder made from the same batch of concrete as the column, approximately equal to the ultimate stress of the concrete of the column,* is a means of comparison of the various columns, and forms a base from which to measure, in percentage, the strength of the individual column.

Applicable test results on only twelve columns are available, and these results, together with the data on the columns, are given in Table 1.

From the length-diameter ratio and the ratio of the strength of the column to that of the test cylinder for each column of the group, the relation between length and strength can be determined. These points are shown on Fig. 2. The representative line passes the 100% abscissa at the ordinate 8.3, which signifies that a small test specimen with the ratio, $\frac{L}{D} = 2$, will develop the same ultimate stress as a column 8.3 diameters long. Theoretically, the line should cross the 100% abscissa at the value of $\frac{L}{D} = 2$, as the ratio for the small test specimen has that value. The discrepancy is probably due to the

* See Sub-section B, p. 1078.

TABLE 1.—PLAIN CONCRETE COLUMNS.

Source.	Diameter, in inches.	Slenderness ratio, $\frac{L}{D}$.	Ultimate Stress, in Pounds per Square Inch.		Ratio: Columns to cylinders.
			Cylinder, average.	Column.	
M.*.....	14	4.2	2 084	2 045	97.5
				2 137	102.4
				2 087	100.1
".....	14	8.5	1 740	1 698	97.8
				1 836	105.5
".....	14	16.6	2 272	1 676	73.6
				1 819	80.0
W466†	10.5	9.72	2 600	2 660	102.4
".....	"	"	2 400	2 660	111.0
".....	"	"	2 250	2 480	110.0
A. C. I.‡....	20.2	7.2	2 775	2 990	107.9
".....	"	"	2 518	2 500	99.3

*"Tests on Concrete Columns, Plain and Reinforced," at Lehigh Univ., 1915, by F. P. McKibben Assoc. M. Am. Soc. C. E., and A. S. Merrill, *Proceedings*, Am. Concrete Inst., 1916, p. 200.

†"Tests on Concrete Columns," by Prof. Owen Morton Withey, Univ. of Wisconsin, *Bulletin 466*, Eng. Series.

‡"Comprehensive Tests for the Institute at the Pittsburgh Branch, U. S. Bureau of Standards," *Proceedings*, Am. Concrete Inst., February, 1915.

larger diameter of the test column, the unit strength being a function of the relative size of the test specimen and the average size of the largest aggregate. That this is so can be demonstrated by considering the extreme case in which the size of the large aggregate is equal to the diameter of the test specimen. It can readily be seen that such a test specimen would suffer a great change in strength due to the large size of the aggregate.

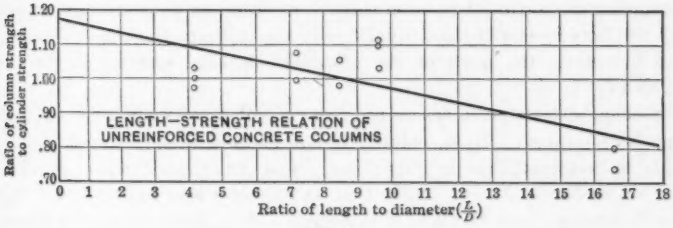


FIG. 2.

The equation of the length-strength line previously obtained, and expressed in Equation (1), is:

$$U_0 = 1.17 - 0.020\,\frac{L}{D} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

which can be expressed in the form of a reduced equation similar to Equation (2):

$$U = 1.00 - 0.0171\,\frac{L}{D} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

If the ultimate strength of the concrete as developed in test cylinders is designated by f'_c, then the ultimate stress of the same concrete in a column will be given by Equation (5), as follows:

$$\frac{P_m}{A} = 1.17 f'_c \left(1.00 - 0.0171 \frac{L}{D}\right) \quad \text{.....(5)}$$

in which,

P_m = total ultimate load, in pounds, on the column;
A = cross-sectional area of the column, in square inches;
f'_c = ultimate strength of the concrete in small test cylinders;
L = length of the column, in inches; and
D = diameter of the column, in inches.

7.—Cast-Iron Columns

The stress-strain diagram for cast iron is of the same general type as that for concrete. Concrete gives warning of impending failure by spalling, the failure itself being of the nature of a combined shear and crumbling. Cast iron gives no notice or warning of impending failure, which is exceedingly violent, almost explosive in character, fragments, sometimes quite large, being violently thrown about.

Like concrete, cast iron is variable in strength. (See Section III.) Uneven cooling, sand holes, blow-holes, internal cold shuts, and unfused chaplet stems, are all likely to occur and also to have an enormous effect in reducing the strength of a cast-iron element, such as a column. By skillful foundry work, allowing proper thickness of metal, and, in general, by good design, cast-iron columns may develop great reliability and strength, and there are being made commercially in the United States columns of such excellence that a comprehensive series of tests on them is in order, as their strength and reliability are far superior to those qualities in the columns that have already been tested and form the basis of the allowable working stresses in use. (See Section IV.)

Frequently, test specimens in the form of cubes are made to determine the strength of the cast iron in columns. For reasons already stated, it is inadvisable to test materials in this form. The American Society for Testing Materials recommends a specimen 1 in. in diameter and from 2.5 to 4 in. long for compression tests of all cast materials. Specimens at least 2 diameters long are excellent for determining the true strength of cast iron, although a small specimen such as this will probably be homogeneous and not have any of the faults of the large castings, and will develop a higher strength than the same metal cast into a column.

Table 2 gives the data on a representative group of cast-iron columns tested by the New York Building Department in 1897.* Although tested so long ago, this group seems to be one of the last, if not the latest series of tests, it being assumed that the tests made during the Nineteenth Century gave results that were final for all time for cast iron. Although data on other tests are available, this small group is sufficiently large for the present purpose, as no

* *Engineering News*, January 13 and 20, 1898.

detailed analysis of cast-iron columns is intended, the group being used merely as a means of comparison with the various types of concrete columns.

Fig. 3 gives the results of the tests showing the variation of strength with length, and the line representing the equation,

$$\frac{P_m}{A} = 30\,500 - 165\,\frac{L}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

derived by William H. Burr, M. Am. Soc. C. E., from the combined results of the Watertown tests and another group tested by him at Phœnixville, Pa., in 1898.* Equation (7), for cast-iron columns, which is similar to Equation (2), is obtained by dividing Equation (6) through by 30 500, the result being,

$$U = 1.000 - 0.0055\,\frac{L}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

TABLE 2.—CAST-IRON COLUMNS.†

Slenderness ratio, $\frac{L}{D}$	Diameter, in inches.	THICKNESS OF COLUMN WALL, IN INCHES.			ULTIMATE STRESS, IN POUNDS PER SQUARE INCH.		Ratio: Actual to theoretical.
		Maximum.	Minimum.	Average.	Actual.	Theoretical‡.	
12.7	15	1	1	1	30 830	28 470	1.083
"	15	$1\frac{5}{16}$	1	$1\frac{1}{8}$	27 700	"	0.973
"	15	$1\frac{1}{4}$	1	$1\frac{1}{8}$	24 900	"	0.873
"	$15\frac{1}{8}$	$1\frac{7}{32}$	1	$1\frac{1}{8}$	25 200	"	0.885
"	15	$1\frac{11}{16}$	1	$1\frac{11}{16}$	32 100	"	1.126
"	15	$1\frac{1}{4}$	$1\frac{1}{8}$	$1\frac{3}{16}$	40 400	"	1.420
20.0	$7\frac{3}{4}$ to $8\frac{1}{4}$	$1\frac{1}{4}$	$\frac{5}{8}$	1	31 900	27 300	0.832
"	8	$1\frac{9}{32}$	1	$1\frac{3}{64}$	26 800	"	0.982
"	$6\frac{1}{16}$	$1\frac{5}{32}$	$1\frac{1}{8}$	$1\frac{9}{64}$	22 700	"	1.167
"	$6\frac{31}{32}$	$1\frac{1}{8}$	$1\frac{1}{16}$	$1\frac{7}{64}$	26 300	"	0.963

† Kent's "Mechanical Engineer's Handbook", 1910 Edition, p. 275.

‡ From Professor Burr's equation: $\frac{P}{A} = 30\,500 - 160\,\frac{L}{D}$.

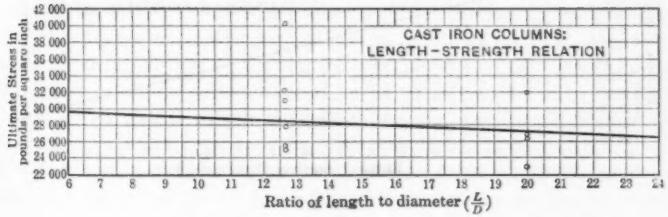


FIG. 3.

8.—STRUCTURAL STEEL COLUMNS

For structural grades of steel, the yield point is 35 000 lb. per sq. in., and the ultimate strength, 55 000 lb. per sq. in., both figures being approximate. In the compression of short specimens, length equal to twice the diameter, a curve the same as the tension curve is produced. However, for any compression

* *Engineering News*, June 30, 1898.

member of a commercial structure, the length is sufficiently great to cause failure through buckling of the member at the yield-point stress.

Table 3 and Fig. 4 give the data on a set of ten structural steel columns tested by Professor Talbot.* These columns were duplicates of steel columns that were cast into concrete columns, as reinforcement for the concrete, or *vice versa*. The columns were excellently fabricated, tested with the usual precision and thoroughness of Professor Talbot, and, as a group, are exactly fitted for comparison with the other types of columns discussed in this paper. The steel, tested in two 32-in. sections of the angles as used in the columns, showed an ultimate compressive strength of 39 800 lb. per sq. in.

TABLE 3.—STRUCTURAL STEEL COLUMNS.

Length.	Cross-sectional area, in square inches.	Slenderness ratio, $\frac{L}{R}$.	Ultimate Stress, in Pounds per Square Inch.		Ratio: Actual to theoretical column strength.
			Actual.	Theoretical.†	
2 ft. 0 in.	13	6.1	37 450	36 180	1.035
4 ft. 8 in.	"	14.4	33 700	34 800	0.968
"	"	"	34 700	"	0.997
10 ft. 0 in.	"	30.8	31 600	32 080	0.985
"	"	"	32 700	"	1.019
"	"	"	32 600	"	1.016
15 ft. 4 in.	"	47.2	28 300	29 350	0.964
"	"	"	28 900	"	0.981
19 ft. 4 in.	"	59.2	28 800	27 370	1.050
"	"	"	26 500	"	0.968

† From the formula: $\frac{P}{A} = 37\,200 - 512\,\frac{L}{D}$, the equation of the line representative of the ten tested columns. D is the distance back to back of angles, and is approximately the column diameter.

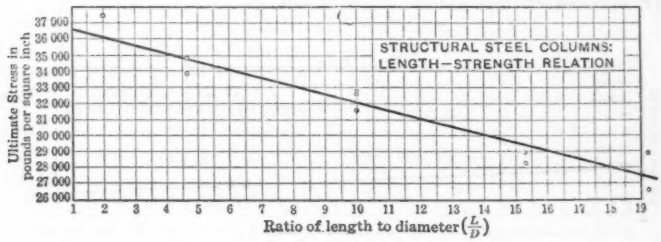


FIG. 4.

The equation for the strength of the columns is:

$$\frac{P_m}{A} = 37\,200 - 167\,\frac{L}{R} \qquad (8)$$

or,

$$\frac{P_m}{A} = 37\,200 - 512\,\frac{L}{D} \qquad (9)$$

* Described in *Bulletin No. 56*, Univ. of Illinois Eng. Experiment Station.

in which,

P_m = the ultimate strength of the column, in pounds;
A = the cross-sectional area, in square inches, of the steel in the column;
R = the radius of gyration of the cross-section of the column; and
D = the diameter of the column (back to back of angles).

Dividing through by 37 200, the comparison equation in the form of Equation (2) is obtained:

$$U = 1.000 - 0.0139 \frac{L}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Professor Talbot gives the formula:

$$\frac{P_m}{A} = 36\,500 - 155 \frac{L}{R}$$

neglecting the 2-ft. specimen, and, apparently, deriving the equation graphically. Equation (8) is determined by means of the normal equations of the theory of least squares, thus eliminating all personal and other errors present in the graphical method, and gives an incontrovertible result.

9.—Reinforced Concrete Columns

A.—Columns Reinforced with Structural Steel Columns

For simplicity, this type of column will be called the steel and concrete column.

The question as to what percentage of steel reinforcement marks the division between the two groups of columns, one of which may be considered as a concrete column reinforced with steel, and the other a steel column reinforced with concrete, has engaged considerable attention. The allowable working stress is dependent on the group into which the column was placed. (The percentage selected as the division was approximately 4.) For this analysis, the columns have been grouped as follows: Those the reinforcement of which will stand compression stress unaided as an individual column, the ultimate stress being about 35 000 lb. per sq. in., and those the reinforcement (longitudinal rods) of which will not stand an appreciable load when tested independently. The results of the analysis prove that the steel will develop the same strength in either case, and, therefore, the division can be eliminated.

The reinforcing element of the steel and concrete column is in itself an independent structural element, capable of functioning as a column, and can be tested in duplicate to determine its individual strength. The concrete will bear the same load as an unreinforced column of the same cross-section. The determination of the load borne by the reinforced column should be simple, being apparently the sum of the loads borne by the two materials independently. The only possible error would be in using a concrete with an ultimate strain of less than 0.0015, at which strain, structural steel in compression exerts its ultimate or maximum strength. For the longitudinal rod reinforced column this would result in less strength being developed by the reinforced column than the sum of the strengths of the steel and concrete.

The results of tests on twelve columns of the type under discussion have been given by Professor Talbot.* Table 4 gives the data on these columns. The twelve structural steel columns used as reinforcement are from the same series of tests and are duplicate specimens of the steel columns described previously under "Structural Steel Columns". As a result of these tests, Professor Talbot draws the conclusions that the strength of the column is equal to that of the structural steel element plus the strength per unit area of concrete equal to the ultimate strength of the concrete as tested in 8 by 16-in. test cylinders.

TABLE 4.—CONCRETE REINFORCED WITH STRUCTURAL STEEL.

Length, in feet.	Ultimate Stress, in Pounds per Square Inch.					Ratio: Column strength, actual to theoretical.
	Test cylinder.	Average cylinder.	Theoretical steel.†	Theoretical column.‡	Actual column.	
4.67	1 350 1 490	1 420	34 800	4 998	4 805 5 010	0.961 1.002
10.00	1 350 1 490	1 420	32 080	4 708	4 236 4 860	0.900 1.032
15.33	1 140 1 260	1 200	29 350	4 210	3 893 4 428	0.925 1.052
19.40	970 1 150	1 060	27 370	3 864	4 076 4 112	1.055 1.064
10.00	2 420 2 520	2 470	32 080	5 640	5 290 5 440	0.938 0.964
10.00	700 660	680	32 080	4 047	4 290 4 407	1.060 1.089

† From the representative equation : $\frac{P}{A} = 37\,200 - 512\frac{L}{D}$.

‡ See text for derivation of theoretical column strength.

Professor Talbot considers the strength of the steel in the steel and concrete column as the average of the strengths of the two duplicate steel columns of the same length, that were tested. Although the results thus obtained are good and show close agreement between the strength of the steel and concrete column and the sum of the strengths of the concrete and the steel column, better agreement between these two quantities can be obtained. The strength developed by the steel in the steel and concrete column is unknown. A better estimate can be made of its strength by deriving the strength from the entire group of ten steel columns than by assuming it to be equal to the average strength of the one, two, or three steel columns of the same length as the steel and concrete column in question. Therefore, the strength of the steel in the steel and concrete column should be taken as that indicated by Equation (8), the straight line representing the strength of the ten steel columns of different lengths.

To emphasize the exactness with which the strength of the columns coincides with the theoretical strength thus determined, the necessary quantities have been calculated and tabulated in Table 4. The difference between the averages of the theoretical and the actual strengths of the group of reinforced concrete columns is 0.37%, thus proving the validity of the original hypothesis and showing the care with which the columns were made and tested.

* *Bulletin No. 56*, Univ. of Illinois Eng. Experiment Station.

From the length-strength equation of the columns of this type, Equation (11), in the form of Equation (2), is obtained:

$$U = 1.000 - 0.0126 \frac{L}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

The length-strength relation cannot be obtained directly from the results given in Table 4 as the concrete in the different length columns was of different strengths. Equation (11) is obtained from theoretical column strengths, using one strength of concrete.

B.—Concrete Reinforced with Longitudinal Rods

This type of reinforcement differs from the structural steel type in that the rods, tested alone, cannot support an appreciable load. There is a wide difference of opinion between testing engineers as to the value of this type of reinforcement. Although almost generally accepted as in the average increasing the strength, there is still an opinion that the rods, by their action in the testing machine, of buckling outward and breaking off the concrete covering, hasten column failure. Even those engineers who most favor this type of column, cannot agree on the value of the rods nor can they prove that they have a definite value. When a rodded column shows low strength, it can be attributed to the concrete, or to the rods. The only rational method of settling the question is to draw logical and warranted conclusions from a mathematical analysis of a group of such columns. The following analysis includes all the available test specimens.

With the longitudinal rod reinforced column, there are two variable quantities: The strength due to the concrete and that due to the rods. The strength of the concrete in a column differs from that of similar concrete in a test cylinder, due to the natural variation in the strength in concrete. This variation is so great that for one specimen, or even for several, no conclusion can be reached as to the strength developed by the rods, as the strength of the concrete in individual columns cannot be determined. If the number of columns analyzed is sufficiently large, the variation between the average strength of the concrete in the cylinders and that in the columns can be eliminated. This principle can be illustrated as follows: Suppose two test cylinders are made from each of various batches of concrete of different proportions and strengths. The strengths of the two cylinders from each batch will differ and show no apparent relation. If, say, twenty batches are taken, the average of the two sets of cylinders will come into close agreement. The larger the number of batches, the closer the agreement between the averages of each set of cylinders.

Mathematical Determination of Rod Strength.—The ultimate column stress for a longitudinally steel reinforced column (neglecting length, which will be treated subsequently, all the columns being considered for the present as of the same length), is evidently of the form:

$$\frac{P_m}{A} = S r_r + f'_o (1 - r_r) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

in which,

P_m = the total ultimate load on the column;
A = the cross-sectional area of the column;
r_r = the ratio of steel rod reinforcement;
f'_o = the stress in the concrete at the ultimate strength of the column; and,
S = the stress in the steel reinforcement at the ultimate strength of the column.

$(1 - r_r)$ is evidently the ratio of concrete. f'_o must be the ultimate strength of the concrete, as in a rodded column, the rods, unsupported, cannot carry any load; that is, the strength of the rods at maximum column strength must be exerted while the concrete is unbroken. The rods, although increasing the strength of the column, cannot increase the strength of the concrete.

Equation (12) can be written in the form :

$$S = \frac{P_m}{A\, r_r} - \frac{f'_o}{r_r}(1 - r_r) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

From the results of column tests, all the values of the variables in Equation (13) are obtained with the exception of S, which, therefore, is determined, being a one valued explicit function of the other variables. The result, S, determined from any individual column is valueless, as it depends on the actual strength of the concrete in the column, which is not known.

However, by increasing the number of test specimens, the average of the ultimate strengths of the concrete in the test cylinders approaches the average strength of the concrete in the column. Consequently, the average of the values of S obtained from a number of individual columns will approach the strength of the reinforcing rods at the ultimate strength of the column.

Equation (13) can be written as follows:

$$S = \frac{P_m}{A\, r_r} - f'_o\left(\frac{1}{r_r} - 1\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

For N columns, there are N equations, the summation of equations of the form of Equation (14), being:

$$S = \frac{1}{N}\sum_N \frac{P_m}{A\, r_r} + \frac{1}{N}\sum_N f'_o\left(1 - \frac{1}{r_r}\right) \quad \ldots\ldots\ldots (15)$$

Now,

$$f'_{om} = f'_{cm} + x$$

in which,

f'_{cm} = the ultimate stress of the concrete of Cylinder M;
f'_{om} = the ultimate stress of the concrete of Column M;
x = therefore, the difference between the ultimate stress of the concrete in the columns and the ultimate stress of the concrete in Cylinder M.

Equation (15) may then be written as follows:

$$S = \frac{1}{N}\sum_N \frac{P_m}{A\, r_r} + \frac{1}{N}\sum_N \left(f'_{cm} + x\right)\left[1 - \frac{1}{r_r}\right] \quad \ldots\ldots (16)$$

therefore,

$$S = \frac{1}{N}\sum_N \frac{P_m}{A\,r_r} + \frac{1}{N}\sum_N f'_{cm}\left(1-\frac{1}{r_r}\right) + \frac{1}{N}\sum_N x\left(1-\frac{1}{r_r}\right). \quad (17)$$

As already stated, the average ultimate strength of the concrete cylinder is equal to the average ultimate stress of the concrete column, or these two averages approach each other. Therefore, $\sum_N x$ approaches zero, and can be neglected in a set of columns as large as that included in this analysis. In consequence, $\frac{1}{N}\sum_N x\left(1-\frac{1}{r_r}\right)$ is very small or practically zero. Therefore:

$$S = \frac{1}{N}\sum_N \frac{P_m}{A\,r_r} + \frac{1}{N}\sum_N f'_{cm}\left(1-\frac{1}{r_r}\right) \quad (18)$$

From Equation (18), it may be noted that the stress in the steel at ultimate column stress can be determined closely from the following quantities:

P_m = the ultimate strength of the column;
A = the cross-sectional area of the column;
r_r = the ratio of rod reinforcement; and
f'_c = ultimate stress of the concrete of the same batch as the column concrete moulded into a cylinder.

TABLE 5.—LONGITUDINALLY REINFORCED CONCRETE COLUMNS.

Source.	Column diameter, in inches.	Ratio, column length to diameter, $\frac{L}{D}$.	Percentage of longitudinal reinforcement (rods).	Ultimate Stress, in Pounds per Square Inch.				Ratio: Actual to theoretical column strength.
				Concrete strength, average of cylinders.	Column strength.	Apparent stress on rods at ultimate column strength.	Theoretical column strength.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
A. C. I.*	20.2	7.2	1.00	3 206	3 300	12 000	3 490	0.945
			0.98		4 200	105 000	3 480	1.207
			2.04	3 256	3 620	21 300	3 819	0.948
			2.02		3 760	28 500	3 812	0.986
			4.08	2 887	4 565	44 000	4 035	1.132
			4.07		4 500	42 500	4 030	1.117
T.†	9 by 9 (square)	14.5	1.55	1 150	1 280	9 550	1 597	0.801
		10.6	1.48	2 004	2 335	24 400	2 432	0.960
		10.6	1.49	1 368	1 607	17 450	1 839	0.874
W_{300}‡	10.0	9.8	2.35	1 960§	2 310	17 000	2 638	0.875
			2.35		2 310	17 000	2 638	0.875
			2.35		2 695	33 400	2 638	1.022

*Am. Concrete Inst., *Proceedings*, February, 1915.

†Univ. of Illinois, *Bulletin 10*, Eng. Series.

‡Univ. of Wisconsin, *Bulletin 300*, Eng. Series.

§Equivalent cylinder strength, equal to strength of concrete in columns. Test cylinders are the best available indication of the strength of the concrete in the individual column. When it is evident that the cylinder strengths in any series of tests for whatsoever reason are not equal to the strength of the concrete in the column, then this latter strength may be determined in some better way than being equal to test cylinder strength. The ratio of the cylinder strength and unreinforced concrete column strengths in the W_{300} series of tests are at variance with the ratios obtained in all other series, including the supplementary W_{466} group. In all other groups of tests, the column and cylinder strengths are very nearly equal; in the W_{466} group the average column strength is 7% higher than the cylinder strength. In the W_{300} series, the average column strength is only 0.874 of the average cylinder strength. The value given in Table 5 is, therefore, taken as 0.874 of the cylinder strength.

Table 5 gives the data on the group of columns analyzed. Applying Equation (18) to this group of columns, the value of S obtained is 31 000 lb. per sq. in., which means that the average stress in the steel reinforcing rods at ultimate column stress was likewise 31 000 lb. per sq. in.

Using the value of 31 000 lb. per sq. in. for the rods, and the ultimate stress for the concrete in the cylinders as that of the concrete in the columns, the percentage of the theoretical to the actual column strength was calculated, with the results shown in Column (9) of Table 5. These values were plotted with the values of $\frac{L}{D}$ for the columns, the line representative of these points giving the reduction in strength with length for this type of column. Assuming that the value of the stress in the rods, at ultimate column stress, varies as the strength of the column as a whole, the rods have a strength of 39 700 lb. per sq. in. for a column of infinitely small length. This value agrees well with the ultimate compressive stress of short sections of the rods tested independently and is conclusive evidence that the rods develop their full ultimate compressive stress (yield-point stress) in columns of this type, corrected for variation in the strength of the column with length.

The length-strength equation for the longitudinally reinforced column, in the form of Equation (1), is:

$$U_0 = 1.28 - 0.034 \frac{L}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

Equation (19) reduces to:

$$U = 1.00 - 0.0265 \frac{L}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

It can be seen by comparing Equation (20) with the similar equations for the other types of columns that the reduction in strength with length for this type of column is rapid.

C.—Concrete Reinforced with Spirals*

Load applied to concrete reinforced with spirals produces secondary stresses in the spiral. A longitudinal compression in the concrete produces a tension in the spiral. The increase in ultimate strength of concrete thus reinforced is due to the reaction of the spirals to the tension stresses produced in them. Due to the indirect or inverse manner in which the spiral adds strength to the column, engineers were skeptical about accepting spiral reinforcement. Numerous tests, however, have demonstrated that spirals do add great strength to concrete; and spiral reinforcement, if used in conjunction with longitudinal rod reinforcement, is generally a recognized method of reinforcement. Many engineers have yet to be converted to the use of spiral reinforcement, and especially to the value of longitudinal rods in spirally reinforced columns. This is brought out by the 1917 Specifications, which designate the necessary percentage of rods as from 1 to 5, without allowing a higher working stress in a column with the 5% of rods than in one with the 1% of rods.

* See Section II for a complete discussion of the internal behavoir of spirally reinforced concrete under load.

The concrete outside the spiral, necessary for fire protection, begins to scale and becomes useless for taking load at the ultimate strain of the concrete (about 0.0015 in. per in.). As the ultimate strain of spirally reinforced concrete is much more than that of unreinforced concrete, the shell contributes no strength to the column at ultimate load. Consequently, in the following analysis, the column will be considered as having no concrete outside the spiral.

No laws showing the relation between the increase of the strength of concrete due to spiral reinforcement and the percentage of spiral, ultimate strength of spiral, etc., have been propounded. All investigators have assumed that spiral reinforcement adds a certain definite unit strength to that of the concrete, irrespective of the strength of the concrete. (Professor Withey deduces an exception to this from the fact that concrete having a strength of 4 900 lb. per sq. in. or more will not have its strength noticeably increased by spiral reinforcement.) It is also generally understood that high-carbon steel wire as spiral reinforcement gives more strength to the concrete in a column than wire of low carbon or structural grade steel.

That spiral reinforcement should add a certain unit strength does not seem the simplest nor the most logical assumption in attacking the problem. It appears more logical to assume that the spiral increases the strength of the concrete in the column by a certain function of the strength of the unreinforced concrete. A constant multiplier is the simplest function to assume; that is, that the increase in strength of the concrete for a fixed percentage of spiral is a definite percentage of the strength of the concrete.

This is expressed by Equation (21):

$$f'_{cs} = (1 + k\, r_s)\, f'_c \qquad (21)$$

in which,

f'_{cs} = ultimate strength of the spirally reinforced concrete;
f'_c = ultimate strength of the unreinforced concrete;
r_s = ratio of spiral reinforcement; and
k = a constant.

Proceeding on this assumption, the percentage increase in strength of the concrete in the column over the strength of the concrete in corresponding test cylinders has been calculated for the group of columns given in Table 6. The percentage increase has been plotted with the percentage of spiral reinforcement in Fig. 5. A representative straight line has been drawn for the points and it may be noted that all the points with the exception of the two *M* groups, 1, 2, and 3, and 14, 15, and 16, apparently are in agreement with the line. It must be remembered that the strength of the columns will vary greatly due to the variation in strength of the concrete and that, therefore, the points representing individual columns will not coincide with the representative line. The agreement of individual points and the representative line cannot be closer than the strength agreement of individual concrete test specimen strengths with the average strength.

The specimens ($W_{466} = 27$, and $W_{466} = 28$) were made from concrete having an ultimate strength of about 4 900 lb. per sq. in. As there were

other columns made by the same investigator in the same group, of concrete of lower strength, that had their strength increased according to the line of Fig. 5, it may be concluded that concrete of such high strength does not gain appreciably in strength due to spiral reinforcement. The specimens made of concrete having a strength of about 4 000 lb. per sq. in., show a slightly lower strength than that indicated from the curve, allowing for possible variation in strength of concrete. At some point, possibly between 4 000 and 4 900 lb. per sq. in., the additional strength due to spiral reinforcement ceases to be proportionately as large as that of concrete of lower

TABLE 6.—SPIRAL REINFORCED CONCRETE COLUMNS.

No.	Source.	Column dimensions (of core): diameter × length, in inches.	Ratio: length to diameter, $\frac{L}{D}$	Spiral Reinforcement. Amount, percentage.¶	Spiral Reinforcement. Yield-point strength, in pounds per square inch.	Spiral Reinforcement. Ultimate strength, in pounds per square inch.	Test Cylinder, Ultimate Strength, in Pounds per Square Inch. Individual.	Test Cylinder, Ultimate Strength, in Pounds per Square Inch. Average.	Columns, ultimate strength, in pounds per square inch.	Ratio: Column to cylinder strength.	Ratio: Actual column to theoretical column strength.
(1)							2 785		3 321	1.69	1.34
(2)	M *	14 by 120	8.6	0.46	70 000	137 000	1 273	1 965	3 280	1.67	1.32
(3)							1 838		3 415	1.85	1.47
(4)							2 040		2 330	1.32	1.05
(5)	W_{466}†	10 by 102	10.2	0.50	81 000	110 000	1 460	1 750	2 140	1.22	0 96
(6)							1 600		2 503	1.56	1.05
(7)	T‡	12 by 120	10.0	0.85	60 000	90 000	1 700	1 650	2 506	1.47	0.99
(8)							1 350		2 080	1.54	1.04
(9)	T‡	12 by 120	10.0	0.845	38 000	53 000	1 500	1 433	2 203	1.47	0.99
(10)							1 450		2 220	1.53	1.03
(11)							2 868		4 760	1.62	1.05
(12)	A §	20 by 144	7.2	0.94	61 000	83 000	3 083	2 945	4 570	1.55	1.01
(13)							2 885		5 210	1.77	1.15
(14)							....		4 900	2.06	1.33
(15)	M *	14 by 60	4.3	0.95	67 000	134 000	2 569	2 374	4 973	2.10	1.35
(16)							2 178		4 595	1.96	1.26
(17)							2 693		4 608	1.48	0.96
(18)	M *	14 by 120	8.6	0.96	67 000	134 000	2 113	3 103	4 593	1.48	0.96
(19)							4 504		5 049	1.63	1.05
(20)							....		2 939	1.36	0.88
(21)	M *	14 by 240	17.2	0.96	67 000	134 000	1 529	2 165	3 602	1.66	1.07
(22)							2 800		4 266	1.97	1.27
(23)	W_{466}†	10 by 102	10.2	1.00	96 000	133 000	1 780	1 770	2 680	1.51	0.96
(24)							1 760		2 600	1.47	0.94
(25)	W_{466}†	10 by 102	10.2	1.00	96 000	133 000	4 060	4 070	5 950	1.47	0.94
(26)							4 080		5 760	1.41	0.90
(27)	W_{466}†	10 by 102	10.2	1.00	96 000	133 000	4 830	4 880	5 760	1.19	††
(28)							4 930		4 920	1.00	
(29)	T‡	12 by 120	10.0	1.64	54 000	73 000	1 150	1 150	2 068	1.81	0.985
(30)	T‡	12 by 120	10.0	1.67	115 000	152 000	1 900	1 900	3 800	2.00	1.02
(31)	T‡	12 by 120	10.0	1.68	54 000	73 000	1 900	1 900	3 793	1.99	1.015
(32)	T‡	12 by 120	10.0	1.71	115 000	150 000	1 800	1 800	3 404	1.89	0.953
(33)	T‡	12 by 120	10.0	1.73	115 000	150 000	1 400	1 400	2 718	1.94	0.976
(34)							3 510				
(35)	M *	14 by 120	8.6	1.95	68 000	95 000	2 318	2 559	5 256	2.06	0.97
							1 850		5 408	2.11	0.995
(36)	W_{466}†	10 by 120	12.0	2.00	98 000	130 000	1 845**	2 003	4 660	2.33	1.10
							2 160**				
(37)	W_{466}†	10 by 120	12.0	2.00	98 000	130 000	1 685**	1 923	4 390	2.28	1.06
							2 160**				
(38)	W_{466}†	10 by 120	12.0	2.00	98 000	130 000	1 765**	1 918	3 660	1.91	0.89
							2 070**				
(39)	W_{466}†	10 by 120	12.0	2.00	98 000	130 000	2 050**	1 885	3 410	1.81	0.845
							1 720**				

* *Proceedings*, Am. Concrete Inst., 1916, p 200.
† *Bulletin No. 466*, Eng. Series, Univ. of Wisconsin.
‡ *Bulletin No. 20*, Eng. Series, Univ. of Illinois.
§ *Proceedings*, Am. Concrete Inst., February, 1915.
‖ *Bulletin No. 300*, Eng. Series, Univ. of Wisconsin.
¶ Based on volume of concrete within spiral.
** Equivalent cylinder strength. (See Table 5.)
†† Omitted from computations. Spiral does not give proportionate increase of strength to this high strength concrete.

strength. As there is only two reinforced columns of concrete of high strength on which to base conclusions, it cannot be decided whether the columns did not show an increased strength due to the high strength of the concrete or to some other property which might be independent of its high strength.

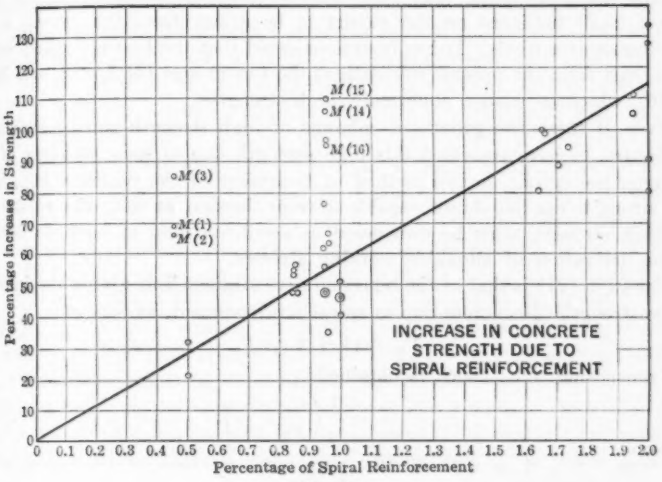


FIG. 5.

The group of columns tested by Messrs. McKibben and Merrill, were made for the purpose of proving and demonstrating the high strength to be obtained with spiral reinforcement. A group thus made is not to be relied on so much as those series made unbiasedly to determine the effect of spiral. The strength of such a spirally reinforced column relative to the strength of the concrete unit is measured by, and is, therefore, a direct function of, the strength of the test cylinders. If by any means the apparent strength of the test cylinders is lowered, then, as a result, the relative and apparent strength of the column is increased, and, therefore, the increase in strength of the concrete in the column due to the spiral, is increased. Several cylinders in this series that failed at high values were discarded by the authors as being abnormal. The practice of arbitrarily discarding discordant results cannot be condemned too severely, as the only legitimate reason for such practice is the proof of that abnormality. If a cylinder tested low, and it could be seen that some defect had caused the reduction in strength, then it would be permissible and proper to exclude the cylinder from all computations. To exclude a cylinder because of high strength, however, is preposterous, as positively no defect could be present. In this analysis, all the test cylinders, including those discarded, are included in the averages in determining the strength of the concrete, resulting in two cases out of the five in bringing the columns (that is, groups of three columns) from marked disagreement into good agreement with the representative line. From this

series of tests, five groups of columns are available for comparison in Fig. 5. Three groups agree with the representative line and the other two groups disagree, showing a much higher strength than the straight line would indicate. A minority of the specimens in a test such as the one under consideration, made for the purpose of obtaining as high a strength as possible, showing an abnormally high and non-concomitant strength, should have only a small influence on the results to be drawn from the several series of columns as a whole. It has been considered that the best procedure would be to omit from the present calculations the two groups (M-1, 2, 3, and M-14, 15, 16), which show such abnormally high strengths.

Even if these two groups of columns of high strength can be shown, in the future, to have possessed this high strength due to some property of the concrete, the spirals, or the method of treatment of the concrete, etc., until this strength can be shown capable of reproduction at will, the reason for the high strength must be considered as accidental and to have no bearing in the derivation of allowable working stresses.

Equation (21) is that of the representative straight line, giving the increase in the strength of concrete due to spiral reinforcement, as follows:

$$f'_{os} = (1 + k\, r_s)\, f'_c$$

Transposing, etc., there is obtained:

$$k = \frac{1}{r_s}\left(\frac{f'_{os}}{f'_c} - 1\right) \qquad (22)$$

The term, $\frac{f'_{os}}{f'_c} - 1$, is 0.01 of the percentage increase of the concrete due to spiral reinforcement. The value of k was calculated for each column, the mean value being 0.573 ± 0.016, and Equation (21) then becomes:

$$f'_{os} = (1 + 0.573\, r_s)\, f'_c \qquad (23)$$

It has been generally understood that the increase in the strength of concrete due to spiral reinforcement is a function of the ultimate strength of the wire constituting the spiral and that wire of high ultimate strength (high-carbon steel) will give superior column strength. This was a natural error to be expected in an initial abstract consideration of the problem. Further, a coincidence in an important group of tests apparently bore out the supposition, although not in as marked a degree as was expected.

Examining all the columns included in Table 6, it may be noted that although the yield-point strength and the ultimate strength of the spirals varied greatly, there is no relation between the variation in strength of these quantities and the increase in strength due to the spiral reinforcement. Even if such a relation existed, it would be difficult to discern, owing to the introduction of the strength variation inherent in concrete.

The group of seven columns tested by Professor Talbot affords an excellent proof of the independence of the increase in column strength and the ultimate strength or yield-point strength of the spiral, and are sufficient, without the consideration of the whole group of spirally reinforced columns, to prove this independence. In analyzing the results of these tests, Professor Talbot assumed that the spiral gives to the column a constant increase in strength

per unit of column area, independent of the strength of the concrete, and it so happened that, in all but three instances, the concrete of low strength was reinforced with a spiral of low strength, and the concrete of high strength was reinforced with a spiral of high strength. The apparent result is to give a greater increase in strength with the spiral of higher strength. In Columns 31 and 32, however, of which the concrete is of the same strength, the spiral with the low ultimate strength caused a higher column strength than that of high ultimate strength. The average value of k from Equation (22) for the columns reinforced with the high-carbon wire differs by less than 1% from its average value for the columns reinforced with the low-carbon wire. The range between the strength of the low-carbon and the high-carbon spiral for this group of seven columns is the extreme for the spirals in all the columns included in Table 6. The constant, k, being a measure of the increase in strength due to spiral reinforcement, this increase is independent of the strength of the spirals for the extreme limits of 60 000 to 150 000 lb. per sq. in. for the ultimate strength and a concomitant variation of from 38 000 to 115 000 lb. per sq. in. in the yield-point stress.

There is, therefore, waste of strength in using high-carbon reinforcement; low steel is preferable as it does the same work, is easier to manufacture, and is not as easily harmed by accidental deformation.

The method of deriving the length-strength equation from the spiral column was similar to that used for the columns reinforced with longitudinal rods; that is, the ratio of theoretical (according to Equation (23)) to actual column strength, expressed in percentage, was determined for each column. The percentages are then plotted with the length of the column, resulting in Equations (24) and (25):

$$U_0 = 1.27 - 0.022\frac{L}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (24)$$

$$U = 1.00 - 0.0173\frac{L}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (25)$$

The general equation of the spirally reinforced column then may be written:

$$\frac{P_m}{A} = 1.27\ (1.00 + 0.573\ r_s)\ f'_c\left(1.000 - 0.0173\frac{L}{D}\right) \ldots\ldots\ldots (26)$$

in which,

P_m = the total ultimate column load, in pounds;
A = the cross-section of the column, in square inches;
r_s = the ratio of spiral reinforcement;
f'_c = the ultimate strength of the concrete, as determined in test cylinders;
L = the length of the column, in inches (or feet); and
D = the diameter of the spiral, in inches (or feet).

Equation (26) is an empirical equation derived as an average for the spirally reinforced column from a set of points. From any set of values, however, even if they are independent of any law of formation, an average

value can be found. Although the points for the individual columns, in a general way, agree with the representative line, they vary to such a large extent from it that the variation could be the result of a fundamental disagreement with the line or from the inherent variation of strength of concrete. The proof that this is the correct equation is neatly applied by the method of measuring the variation in the strength of materials, as developed and described in Section III. The individual variation in strength of all the columns in Table 6 from that given by Equation (23) has been obtained. The measure of this variation is almost the same as it would be if the columns were made of concrete without any reinforcement, showing the variation in the strength of the columns to be due to the inherent variation in the strength of the concrete and proving Equation (23) to be correct.

D.—Columns Reinforced with Spirals and Longitudinal Rods

Authorities differ as to the value of longitudinal reinforcing rods in spirally reinforced columns. It is a common impression that the rods buckle, bending outward, thus bearing on the spiral and causing it to fail. That the rods generally caused an increase in strength was admitted and could be proved from the results of tests. This is a simple process, as the average strengths of two groups of columns that were identical, except that one set had no reinforcement, can be compared, with the result that the group with rods is always the stronger. The contention, however, is and has been, that, although the average is stronger, in an individual case the rods may add no strength, or, worse, decrease it. The inherent variation in the strength of concrete made the problem complex, and it is impossible to measure the strength that the rods developed in any one column. The statement that a low column strength was the result of the rods could be disputed, but not disproved. The method of determining the increase in the strength of the concrete due to spiral reinforcement being at fault added to the confusion.

No method of eliminating the effect of the variation in strength of concrete has ever before been used. The column was considered a unit, the strength of the column as a unit being considered somewhat as a function of the number of rods used as reinforcement. The actual stress developed by the rods, considered as independent units, however, was never attempted.

The strength of the column is a function of the following: The strength of the concrete, the strength added to the concrete by the spiral reinforcement, and the stress in the longitudinal rods at ultimate column strength. In each column, therefore, there are three unknowns. It is known, however, that the spiral increases the strength of the concrete in a definite manner, as given by Equation (23):

$$f'_{cs} = (1 + 0.573\ r_s)\ f'_c$$

Equation (23) is valid when rods are used, as they cannot increase the strength of the concrete.

There are now, therefore, only two unknowns, the strength of the concrete in the column and the stress in the rods at ultimate column strength. As in the rod reinforced column (previous to the development of the formula for the

strength of the rods), the strength of the concrete in an individual column is unknown. Its strength will not be the same as that of the concrete in the test cylinders from the same batch of concrete, and to determine the strength of the rods by the difference between the strength of the column and that due to the spirally reinforced concrete, assuming that the column concrete is of the same strength as the cylinder concrete, leads to absurd results that are sometimes negative. The unit strength of the rods at ultimate column strength thus calculated is given in Column (9) of Table 7. It will be noticed that these quantities are not the true stress in the rods, as that is impossible to determine, and, on account of the large variation in these figures, it is not to be concluded that the strength developed by the rods is such a variable quantity. It will be shown subsequently that the rods develop a very constant strength.

The average of the values given in Column (9) of Table 7 gives the true average value that the rods develop at ultimate column strength, as, in the averaging, the inherent variation in strength due to the concrete is eliminated. This can be understood by considering the demonstration given for columns reinforced with rods. The average of the values in Column (9) of Table 7, is 36 500 lb. per sq. in., which figure gives the impression that it is correct, and that, therefore, the whole basis on which the result was derived was also correct.

Obtaining the length-strength relation by the same method used for the spiral reinforced column, the equation:

$$U_0 = 1.20 - 0.022\frac{L}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

and the reduced equation:

$$U = 1.000 - 0.0183\,\frac{L}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

are obtained.

Assuming that the strength of the rods varies with that of the column as a whole, the value for the rods for a column of infinitesimal length is 43 800 lb. per sq. in. The exact figure for the strength of the rods cannot be obtained, because of the method that must be used. The value, 43 800 lb. per sq. in., however, is proof that the rods exert their yield-point stress at the ultimate column stress, and, as a corollary, it may be stated that, from these results, it is conclusive that the rods do not cause any reduction in the strength of the column or of its components.

The general equation to express the strength of the concrete column reinforced with spiral and longitudinal rods is, as follows:

$$\frac{P_m}{A} = 1.20\left[(1 - r_r)(1 + 0.573\,r_s)\,f'_c + 36\,500\,r_r\right]\left[1.00 - 0.0183\,\frac{L}{D}\right]. \quad (29)$$

in which,

r_r = the ratio of longitudinal steel rod reinforcement;
r_s = the ratio of spiral reinforcement;
P_m = the total load borne by the column at ultimate strength;
A = the cross-sectional area of the column within the spiral;

L = the length of the column;
D = the diameter of the column within the spiral; and
f'_c = the ultimate test cylinder strength.

TABLE 7.—ROD AND SPIRAL REINFORCED CONCRETE COLUMNS.

Source. (1)	Column dimensions (of core): diameter × length, in inches. (2)	Ratio: Length to diameter, $\frac{L}{D}$ (3)	Reinforcement rods, percentage. (4)	Reinforcement spirals, percentage. (5)	Test Cylinder: Ultimate Strength, in Pounds per Square Inch. Individual. (6)	Average. (7)	Columns: Ultimate strength, in pounds per square inch. (8)	Stress on rods at ultimate column strength, in pounds per square inch. ¶ (9)	Ratio of actual to theoretical column strength. (10)
A *	20 by 144	7.2	0.98	0.94	3 396 2 773 2 860	3 009	4 710 4 910 5 140	24 500 43 900 68 400	0.914 0.960 1.000
A *	20 by 144	7.2	0.99	1.88	3 035 2 883	2 959	6 025 6 495	—500 46 900	0.913 0.970
A *	20 by 144	7.2	1.98	0.94	3 185 2 575 2 675	2 812	5 050 4 785 5 885	37 600 24 200 79 800	0.975 0.922 1.144
W †466	10 by 102	10.2	2.00	0.50	2 100 1 400	1 760	3 320 3 280	55 000 53 000	1.150 1.137
W †466	10 by 102	10.2	2.00	1.00	2 100 1 890	1 995	4 050 3 760	48 500 34 000	1.088 1.012
W ‡300	10 by 120	12.0	3.50	2.00	1 825‖ 1 945‖	1 885	4 470	16 300	0.929
W ‡300	10 by 120	12.0	3.50	2.00	1 790‖ 2 170‖	1 980	4 200	2 140	0.843
W ‡300	10 by 120	12.0	3.50	2.00	2 000‖ 1 975‖	1 988	4 970	24 060	0.985
W ‡300	10 by 120	12.0	3.50	2.00	2 160‖	2 160	5 360	25 400	1.000
W †466	10 by 102	10.2	3.77	0.50	2 240 2 120	2 180	4 240 4 080	30 900	1.065 1.025
W †466	10 by 102	10.2	3.77	1.00	1 880 1 720	1 800	4 050 4 340	31 150	1.012 1.085
A *	20 by 144	7.2	6.15	0.95	3 095 2 670 2 775	2 850	6 180 5 750 6 350	33 500 26 500 36 300	0.930 0.861 0.955
W †466	10 by 102	10.2	5.83	1.00	2 020 1 520	1 770	5 760 4 980	53 700 40 500	1.298 1.073
W †466	10 by 102	10.2	6.11	0.50	2 110 2 000	2 055	5 190 5 050	44 500 42 400	1.128 1.098
W †466	10 by 102	10.2	6.11	1.00	1 660 1 710	1 685	4 730 4 580	37 800 34 400	1.067 1.000
W †466	10 by 102	10.2	6.11	1.96	2 270 2 690	2 480	6 510 6 650	25 100 27 600	0.928 0.947
W †466	10 by 102	10.2	6.11	1.00	4 720 4 080	4 400	7 480 7 100	.....	.....
W †466	10 by 102	10.2	6.11	1.00	4 900 4 920	4 910	8 050 8 250	.....	.....
W †466	10 by 102	10.2	8.00	1.00	2 380 2 350	2 365	6 760 7 090	41 700 45 900	1.091 1.143
W †466	10 by 102	10.2	8.00	1.96	2 310 2 470	2 380	7 250 6 680	30 800 23 700	0.966 0.892
W †466	10 by 102	10.2	10.12	1.96	2 280 2 330	2 305	6 190 7 990	17 350 35 100	0.786 0.847

* *Proceedings*, Am. Concrete Inst., February, 1915.
† *Bulletin No. 466*, Eng. Series, Univ. of Wisconsin.
‡ *Bulletin No. 300*, Eng. Series, Univ. of Wisconsin.
§ Based on volume of concrete within spiral.
‖ Equivalent cylinder strength. (See Table 5.)
¶ Not corrected for column length. Average for the total column of values = 36 500 lb. per sq. in.

E.—Columns Reinforced with Spiral and Cast Iron

This general type of column was first suggested and used on the Continent by Dr. von Emperger of Austria.*

Spirally reinforced concrete undergoes considerably more strain at ultimate load than plain concrete, which has an ultimate strain ranging from 0.0006 to 0.0016 in. per in., whereas spirally reinforced concrete has an ultimate strain of 0.0035 to 0.0150 in. per in. (See Section II.) The stress in cast iron of ordinary grade at a strain of 0.0016 is only 16 000 lb. per sq. in whereas for a strain of 0.0035, its stress is about 35 000 lb. per sq. in., and may be as much as 90 000 lb. per sq. in. Morley† gives a stress-strain curve for such a specimen. It is evident, therefore, that a longitudinal structural element of cast iron in a spirally reinforced column would exhibit at least the strength of structural steel, and considerably more than it would in a plain concrete column. With the better grade of castings that are being introduced in the United States, there is reason to expect a new era for the use of cast iron.

TABLE 8.—CONCRETE COLUMNS REINFORCED WITH SPIRAL AND CAST IRON.

Length of column, in feet.	Cast-Iron Reinforcement.		Percentage of Areas of Net Column Area Within Spiral.				Column Load, in Pounds.		Ultimate Stress, in Pounds per Square Inch.	
	Diameter outside, in inches.	Thickness, in inches.	Spiral.	Rods.	Cast iron.	Concrete.	At first sign of failure.	Maximum (ultimate).	Column (mean).	Cylinder.
6	6	¾	0.71	0.68	12.7	86.6	436 500	1 057 000	10 880	4 380
6	6	¾	0.71	0.68	12.7	86.6	776 000	1 026 000	10 560	4 380
8	6	¾	0.71	0.68	12.7	86.6	436 500	940 000	9 680	3 720
8	6	¾	0.61	0.58	10.9	88.5	570 000	1 031 500	9 110	3 720
8	5	1	0.65	0.62	11.9	87.0	630 500	925 000	8 730	4 150
8	7	⅝	0.79	0.76	14.4	84.9	565 500	920 000	10 560	4 150
10	6	¾	0.71	0.68	12.7	86.6	643 500	911 000	9 380	3 750
10	6	¾	0.71	0.68	12.7	86.6	625 000	940 000	9 680	3 750
10	6	¾	0.71	1.26	12.7	86.2	776 000	896 000	9 230	4 300
10	6	¾	0 71	1.26	12.7	86.2	727 500	1 066 000	10 980	4 300
10	(‡)	(‡)	0.61	1.08	11.8	87.0	850 000	1 071 800	9 480	4 300
10	(‡)	(‡)	0.61	1.08	11.8	87.0	576 000	996 000	8 800	4 300
12	6	¾	0.71	0.68	12.7	86.6	732 500	951 500	9 800	4 250
12	6	¾	0.71	0.68	12.7	86.6	630 500	909 000	9 360	3 900
14	6	¾	0.71	0.68	12.7	86.6	630 500	888 500	9 150	4 250
14	6	¾	0.71	0.68	12.7	86.6	679 000	837 500	8 520	4 400
8	6	¾	....	....	100.0	....	480 000	480 000	39 000	.....
8	6	¾	....	....	100.0	....	540 000	540 000	43 800	.....

‡Special I-section.

Table 8 and Fig. 6 give the data on a group of sixteen columns of the von Emperger type. These columns were tested at the Pittsburgh Branch of the U. S. Bureau of Standards for L. J. Mensch, M. Am. Soc. C. E., and have been described by him.§

The cast-iron reinforcement was of two types, one being a hollow tube or cylinder, the other having an I-section. The hollow-core type is advan-

* *Beton und Eisen*, 1912, p. 118.

† "Theory of Structures."

§ "Tests of Concrete Columns with Cast-Iron Reinforcement," *Proceedings*, Am. Concrete Inst., 1917, p. 22

tageous in that it gives a larger radius of gyration and, consequently, greater stiffness and strength for the same cross-section. The hollow core can be used, if sufficiently large, for piping or ventilation. The maximum load is carried by the column at the time the concrete fails, and this occurs before the spirals fail in tension. (See Section II.) If compression of the column is continued, the spiral will begin to fail in tension. Further compression causes the failure of the cast iron inside, which is indicated by a pronounced thud. In consequence of this method of failure, the concrete, at maximum column load, is carrying the same stress as it would have carried if the cast iron had not been used as reinforcement, which is that of the concrete as increased by the spiral reinforcement. The added strength of the von Emperger column is due to the cast-iron core, and this may be found by difference. Mr. Mensch follows this method, using as the strength of the concrete the average ultimate stress of the two concrete columns reinforced with spiral only (and the same percentage of rods, as the remainder of the von Emperger type tested). The concrete in these two columns, however, was much stronger than that used in the von Emperger type of columns, as shown by the strengths of the test cylinders. It would be much better to

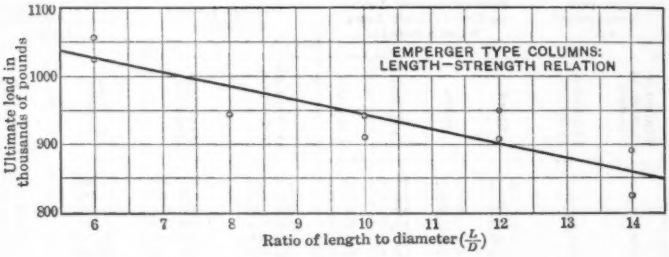


FIG. 6.

use the strength of the concrete in the test cylinders, increased in strength by the spiral, as given by Equation (23). The net result of the latter treatment is to show a smaller stress in the cast iron when the column is bearing maximum load, the value being 37 400 lb. per sq. in., computed by the latter method. This value gives an accurate means of obtaining the strain at which reinforced concrete fails. From the stress-strain diagram of the cast iron similar to that used as reinforcement in the columns mentioned, it is seen that the strain corresponding to the stress of 37 400 lb. per sq. in. is 0.0034 in. per in.

Plotting the strengths of the nine columns of the same cross-section, the length-strength relation obtained is:

$$U_0 = 1.220 - 0.0219 \frac{L}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

or, reduced,

$$U = 1.000 - 0.0179 \frac{L}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

The equation for the column of this type can be expressed as:

$$\frac{P_m}{A} = \left(1.220 - 0.0219\,\frac{L}{D}\right)\left[f'_{os}\,(1 - r_i - r_r) + Ir_i + wr_r\right] \ldots\ldots (32)$$

in which,

P_m = the total ultimate load on the column, in pounds;
A = the total cross-sectional area of the column (not including the space in the core), in square inches;
L = the length of the column, in inches or feet;
D = the diameter of the column, in inches or feet, within the spiral;
r_i = the ratio of cast-iron reinforcement;
r_r = the ratio of longitudinal steel reinforcement;
w = the yield point of the steel reinforcement (this may be assumed to be 35 000 lb. per sq. in.); and
I = the stress in the cast iron at the strain of 0.0034.

f'_{os} is given by Equation (23).

F.—Comparison of the Various Length-Strength Equations

Table 9 is given to enable a comparison to be made between the so-called reduced equations in the form of Equation (2), showing the length-strength relations.

TABLE 9.

Type of Column.	Equation.
Cast iron	$U = 1.000 - 0.0155\,\frac{L}{D}$
Structural steel and concrete	$U = 1.000 - 0.0126\,\frac{L}{D}$
Structural steel	$U = 1.000 - 0.0139\,\frac{L}{D}$
Concrete, no reinforcement	$U = 1.000 - 0.0171\,\frac{L}{D}$
Spiral reinforced concrete, no rods	$U = 1.000 - 0.0173\,\frac{L}{D}$
von Emperger	$U = 1.000 - 0.0179\,\frac{L}{D}$
Spiral and rod-reinforced concrete	$U = 1.000 - 0.0183\,\frac{L}{D}$
Concrete reinforced with rods	$U = 1.000 - 0.0265\,\frac{L}{D}$

SECTION II.—THE INTERNAL BEHAVIOR OF SPIRALLY REINFORCED COLUMNS

1.—Source of Strength

That the ultimate strength of the spirally reinforced column is not due to the spiral alone, the concrete being considered as a substance offering no resistance and, therefore, acting as a perfect fluid, can be proved as follows: The longitudinal column stress and that in the spiral in this case

would be equal, respectively, to the hydraulic pressure in a thin cylinder and the lateral tension in the cylinder walls, the percentage of the volume of the cylinder walls equalling that of spiral reinforcement. The equation giving the relation between the lateral cylinder stress and the hydraulic pressure, transposed to meet the present requirements, gives Equation (33):

$$p = \frac{p_s\, r_s}{2} \qquad (33)$$

in which,

p = the longitudinal stress in the concrete;
p_s = the stress in the spiral; and,
r_s = the ratio of spiral reinforcement.

For a 2% spiral reinforcement of the ultimate strength of 150 000 lb. per sq. in., which values are those to give the largest value to p in Equation (33) for the materials in any or all the test columns examined in Section I, the column stress, p, is found to be 1 500 lb. per sq. in. It is shown, subsequently, that ultimate or preferably maximum column stress occurs when the spiral is stressed at most to its yield point. For a 2% spiral reinforcement stressed to a yield point of 115 000 lb. per sq. in., again values taken from the same group of columns being those to give to p its greatest value, the column stress, p, is found to be only 1 150 lb. per sq. in. These computed values for the column stress are only a fraction of the ultimate stresses actually developed in columns of this type; therefore, the strength of the column is not developed solely by the spiral reinforcement. (See Table 6.) As a corollary, it might be stated that the concrete is not valueless at maximum column load, that it is not completely destroyed, and that it does not act merely as a perfect fluid transmitting the compressive forces to the spiral reinforcement.

2.—Mathematical Equations

Any equation for the spirally reinforced column can be derived from the following five fundamental equations:

$$e_r = \frac{p}{E_c} - \frac{2}{m} \times \frac{p_l}{E_c} \qquad (34)$$

$$e_l = \frac{1}{m} \times \frac{p_l + p}{E_c} - \frac{p_l}{E_c} \qquad (35)$$

$$\frac{r_s}{2}\, p_s = p_l \qquad (36)$$

$$e_l = e_s \qquad (37)$$

$$p_s = e_s\, E_s \qquad (38)$$

in which,

E_c = Young's modulus for concrete (unrestrained);
E_s = Young's modulus for steel (30 000 000 lb. per sq. in.);
p = the longitudinal stress in the column (concrete);

e_r = the longitudinal strain in the column (concrete), the subscript, r, denotes that the strain is for a laterally (by spiral) restrained concrete, and is not equal to the strain as given by the equation, $p = eE_c$;

p_l = the lateral stress in the column (concrete);

e_l = the lateral strain in the column (concrete);

p_s = the stress in the spiral reinforcement;

e_s = the strain in the spiral reinforcement;

r_s = the volume ratio of spiral to concrete within spiral (the percentage spiral reinforcement times 0.01); and

$\frac{1}{m}$ = Poisson's ratio for concrete (unreinforced), that is, the ratio of lateral to longitudinal strain.

The symbol, E_c, requires some explanation. By definition,

$$E_c = \frac{p}{e} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (a)$$

in which, e is the strain resulting in concrete stressed to the intensity, p. E_c is not a constant, but varies with the intensity of p. For values of e equal to or less than the ultimate strain of laterally unrestrained concrete, E_c can be determined directly from the equation, $E_c = \frac{p}{e}$, by using observed values of p and e of unreinforced concrete, a method becoming inoperative beyond the ultimate strain of concrete. The value of E_c can also be obtained from Equation (34) for all strain values, both less than and more than the ultimate strain of unreinforced concrete. Note must be made that the value of E_c obtained by the two methods outlined does not correspond for the identical values of the strain, e, and also that the quantity, E_c, as ordinarily understood as the ratio of stress to strain, is an imaginary quantity for values of e beyond the ultimate strain of unreinforced concrete. In both Equation (34) and Equation (a), E_c has the same 1:1 relationship with p.

Equations (34) and (35) are derived from the study of the strains in a homogeneous material under stresses along the three co-ordinate spatial axes.*

If r_s is the percentage of spiral in a column, and a cut is made through the axis of the column, the ratio of the areas of exposed steel and concrete in the longitudinal cross-section is $\frac{1}{2}r$. The total tension in the spiral must equal the total compression in the concrete, or, $A_s p_s = A_c p_l$. Therefore, $\frac{A_s}{A_c} p_s = p_l$. As $\frac{A_c}{A_s} = \frac{2}{r_s}$, therefore, $\frac{1}{2} r_s p_s = p_l$, which is Equation (36).

The spiral is rigidly fixed to the concrete, and, therefore, any strain in the concrete must be accompanied by an equal but opposite strain in the spiral; therefore, $e_l = e_s$.

Equation (38) is the simple stress-strain relation for steel. This is nearly a constant to the yield point.

* "Strength of Materials", by Arthur Morley, Chapter I.

A.—Derived Equations

Substituting in Equation (34) the value of p_l given in Equation (36), there is obtained:

$$p = e_r E_c + \frac{r_s}{m} p_s \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (39)$$

Further elimination of variables gives,

$$p = \frac{m e_r E_c \left(m \left[\frac{2}{r_s n} + 1\right] - 1\right)}{m^2 \left(\frac{2}{r_s n} + 1\right) - (m + 2)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (40)$$

and,

$$p_s = \frac{2}{r_s} \frac{m e_r E_c}{m^2 \left(\frac{2}{r_s n} + 1\right) - (m + 2)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (41)^*$$

3.—Application for Equations

Equation (39) shows that the strength of the spirally reinforced concrete is due to two practically independent quantities, the first of which, $e_r E_c$, is a function of the concrete, and the second, $\frac{r_s}{m} p_s$, a function of the stress in the spiral and Poisson's ratio, all these values being those at the maximum column stress. Equation (39) can be written:

$$p = (e_r E_c)_m + X + \frac{r_s}{m} p_s \quad \ldots\ldots\ldots\ldots\ldots\ldots (42)$$

in which $(e_r E_c)_m$ is the ultimate or maximum strength of the unreinforced concrete, and, therefore,

$$e_r E_c = (e_r E_c)_m + X$$

At the ultimate strain of unreinforced concrete, approximately 0.0015 in. per in., $e_r E_c = (e_r E_c)_m$. An increase in strain and load beyond this point increases p. In what manner does X vary? This can be determined from Equation (39) by using known and substituted values of, $\frac{r}{m} p_s$, in which,

$p_s =$ 150 000 lb. per sq. in.;
$r =$ 0.02; and
$m =$ 2 (the minimum possible).

All these are values to give the greatest value to $\frac{r}{m} p_s$, which becomes $\frac{150\,000 \times 2}{2 \times 100} = 1\,500$, and which is the maximum that $\frac{r_s}{m} p_s$ can assume. It is probably, as the criterion, larger than the value which it actually does assume. Again, as it can be proved that the spiral at maximum column stress

* In "Materials of Engineering", Johnson has developed a somewhat similar equation for p_s. This equation has been commonly accepted, but it contains one indeterminate quantity, and checks poorly with experiments. Equation (41) gives values which check closely with experiments.

is at most stressed to the yield point, using yield-point values which give p a maximum value, 1 150 lb. per sq. in. is obtained for $\frac{r_s}{m} p_s$. These values are far below the actual increase in strength due to the spiral for a 2% reinforcement and in the same manner can be proved proportionally less for smaller percentages of reinforcement. (See Table 6.) For example, concrete of a strength of 3 000 lb. per sq. in., reinforced with 2% of spiral, would have its strength increased approximately 3 400 lb. per sq. in., according to Equation (23).

Therefore, X must be positive, as,

$$(e_r E_c)_m + \frac{r_s}{m} p_s < p,$$

and $e_r E_c$ must increase beyond the value $(e_r E_c)_m$. The strength of the spirally reinforced column, therefore, must be due to some inherent quality of the concrete which permits it to be compressed uninjured, when spirally reinforced, greatly in excess of the ultimate strain of unreinforced concrete. Equation (39) shows that the greatest part of the increase in strength caused by spiral reinforcement is due to the increase in $e E_c$ over the value $(e E_c)_m$, and that this part of the increase is practically independent of Poisson's ratio and the stress in the spiral. The magnitude of $e_r E_c$ for any value of e_r, is independent of the stress in the spiral, but is a function of the percentage of spiral reinforcement, as empirically determined.

Referring again to the concrete with an ultimate strength of 3 000 lb. per sq. in., reinforced with 2% of spiral and thus increased by 3 400 lb. to the value of 6 400 lb. per sq. in., the following calculations can be made. The largest possible value of $\frac{r_s}{m} p_s$, in Equation (39), is 1 150 lb. per sq. in. The value of e_r at ultimate column strain, approximately 0.0035 in. per in., compares with the value of e_r at the strain of unreinforced concrete of 0.0015 in. per in. From this, it may be noted that the variable, E_c, is much lower at the ultimate column strength than at the ultimate strength of unreinforced concrete.

The lateral strains for a spirally reinforced concrete are shown by extrapolation of direct experimental measurements to be between 0.0006 and 0.0030 in. per in. (See Section II, 4 D.) This is equivalent to stresses of 18 000 and 90 000 lb. per sq. in., respectively, in the spiral. By Equation (38), the maximum increase in column strength due to 2% of spiral reinforcement (that is, the second term of Equation (39), $\frac{r_s}{m} p_s$), stressed to 90 000 lb. per sq. in., is 900 lb. per sq. in., which is the maximum increase in strength due to spiral alone for the group of columns analyzed in Section I. The maximum increase due to the spiral alone, corresponding to a spiral stress of 18 000 lb. per sq. in., is 180 lb. per sq. in. This is a variation of only 720 lb. per sq. in., due to the maximum difference in spiral stress at maximum column load.

The increase in strength of spirally reinforced concrete, therefore, is practically independent of the strength of the spiral reinforcement or the

stress in it at the instant of maximum or ultimate column load, as the maximum variation in the stress in the spiral causes a maximum variation in column strength of only 360 lb. per sq. in. which is approximately 6 per cent. If the stress in the spiral, at ultimate column load, is between 18 000 and 38 000 lb. per sq. in., the latter being the lowest value of the yield point of the spiral, variation in the strength of the spiral in the particular column would not affect the maximum strength of the column, as all spirals would be capable of being stressed to at least 38 000 lb. per sq. in. If the concrete compressed sufficiently to stress the spiral beyond 38 000 lb. per sq. in. then, from Equation (38), it can be seen that the column strength would be slightly influenced by the variation in yield point and consequent variation in ultimate strength of the spiral.

In the group of columns tested at the University of Illinois, however, the increase in strength due to the spiral reinforcement was independent of the yield-point stress of the spiral. This would indicate that, for stresses of 18 000 lb. per sq. in. or more, in the spiral, which is the spiral stress range in which column failure occurs, the rate of decrease in $e_r E_c$ is algebraically as large as that of $\frac{r}{m} p_s$, or, if $e_r E_c$ did not decrease more rapidly than $\frac{r}{m} p_s$, that all the stresses in the spirals for this group of columns at maximum column load were under the lowest yield point of the group of spirals. In this latter case, the strength of the column would not be influenced by the variation in the yield point of the spirals used. This is within the limits of possibility, although very improbable.

4.—Properties and Behavior Under Load

A.—Poisson's Ratio

Poisson's ratio, $\frac{1}{m}$, is that of the longitudinal to the lateral strain in a material, and, therefore, may be written as $\frac{e_h}{e_l}$. For a perfect fluid, for which Poisson's ratio is a constant and also has the greatest possible value, $\frac{d\, e_h}{d\, e_l} = 0.5$ and $\frac{fd\, e_h}{fd\, e_l} = 0.5$. Poisson's ratio cannot be greater than 0.5, otherwise a material could increase in volume due to compressive stress, which is absurd, and leads to a direct contradiction of the principle of the conservation of energy.

Poisson's ratio is not $\frac{d\, e_h}{d\, e_l}$, but is $\frac{fd\, e_h}{fd\, e_l}$. As for all solids, the initial value of $\frac{e_h}{e_l}\left(\frac{1}{m}\right)$ is much less than 0.5, the value of $\frac{1}{m}$ at the failure of the material, $\frac{fd\, e_h}{fd\, e_l}$, even if at that point the material had become a perfect fluid, would be less than 0.5.

For spirally reinforced concrete, Poisson's ratio is smaller than it is for unreinforced concrete. To the ultimate strain of unreinforced concrete the

spiral stress, as obtained both by measurement and by application of Equation (41), is at the most 10 000 lb. per sq. in. Equation (36) shows that 2% of spiral, when stressed to this value, produces a lateral stress in the concrete of 400 lb. per sq. in. (and for lesser percentage of spiral, a lesser effect). Equation (34) shows that the longitudinal compression is reduced 10% by the 2% of spiral reinforcement, and Equation (35) shows that the lateral expansion is reduced 50% by the reinforcement, the net result being a reduction of Poisson's ratio to five-ninths of its unrestrained value.

Several of the longitudinal and lateral strain readings for columns of the groups tested, and described in *Bulletin No. 10* of the University of Illinois and *Bulletin No. 466* of the University of Wisconsin, show the value of $\frac{1}{m}$, computed from readings on identical columns, to increase with the increase in stress, and to show, among possible values, a percentage of such high final values as 0.7, 0.85, etc., an apparent contradiction to the previous statements. On examination of the method of making these measurements, together with the impossibility of measuring exactly Poisson's ratio for a non-homogeneous material such as concrete, the discrepancy between theory and fact vanishes. The lateral strain was measured at one or two places on the column; the longitudinal strain was measured for the whole length of the column. Consequently, the longitudinal strain measured was the average for the whole column, but the measurements of lateral strain showed the value for only one or two particular points on the length of the column. The two quantities were not measured, therefore, for the same series of particles of concrete, only one or two particles being common to the two measurements. If the strains were constant throughout the mass of the concrete, this method would be valid, but with a non-homogeneous substance such as concrete, the variation in strains throughout the mass makes such measurements valueless as, for example, the present values obtained from Poisson's ratio. Although the magnitude of $\frac{1}{m}$ is valueless, the manner in which it varies with variation in load can at least be placed to some use, as it indicates qualitatively, if not quantitatively, the variation of the true value of Poisson's ratio with increasing stress.

In justice to the authors of the two *Bulletins* mentioned, it must be stated that they realized the fallacy of such figures and did not include them in the values which they measured and computed for Poisson's ratio. Professor Talbot mentions the inadequacy of the results. Messrs. McKibben and Merrill include the three astounding values of 0.82, 0.563, and 0.620, in their measurements.

Fig. 7 gives several representative curves for the so-called Poisson's ratio, derived from the one-place lateral measurements. It can be seen that $\frac{1}{m}$ always increases with increase in load, but in no general manner. Some curves are nearly straight lines, whereas others are similar to the stress-strain curves for spirally reinforced concrete, having a rather sudden change of direction, with a long continuation of the curve.

B.—Ultimate Strain for Spirally Reinforced Concrete

Professors Withey and Talbot have made numerous direct measurements of the strain of spirally reinforced concrete columns, several representative curves being given in Fig. 8. For obvious reasons, the ultimate strain cannot be determined precisely in this manner. The strains are very small linear measurements, even for such comparatively large strains as occur at the failure of spirally reinforced concrete. The shell has been sheared, and the surface

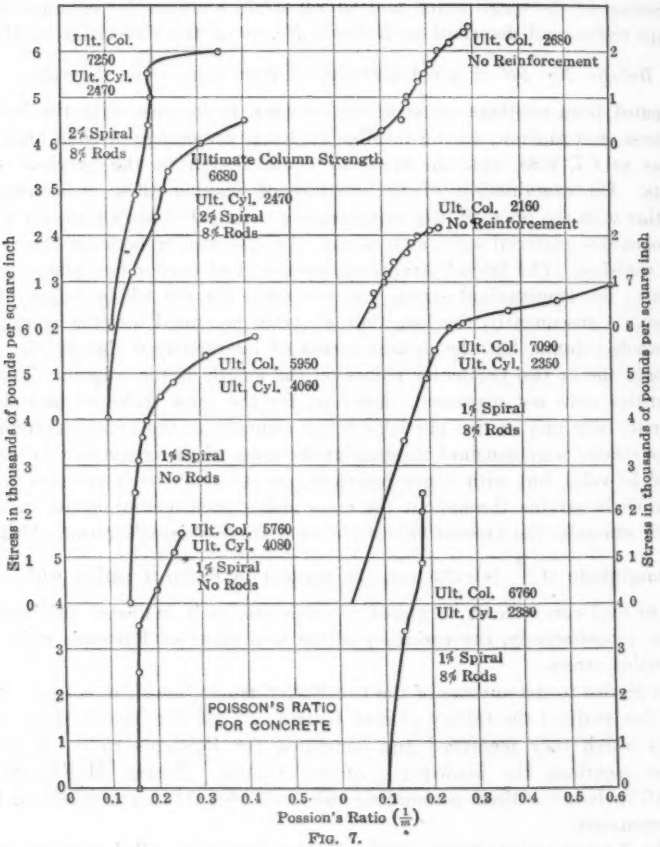


FIG. 7.

concrete is in a state of spalling. Such conditions are not conducive to good results. Nevertheless, observations by such excellent technicians at least show the stress-strain relations to and near failure and give good indications of column behavior at the point of failure. Several observations show clearly the strain at which maximum (ultimate) stress was developed. The stress at that point was a true mathematical maximum, the stress values for points on

curves passing through the point being less than the stress value at the point for all values of the strain lesser and greater than the strain value at the point. In these cases, the strain was carried a considerable distance beyond the strain at maximum load. It may or may not be that the cause of the maximum

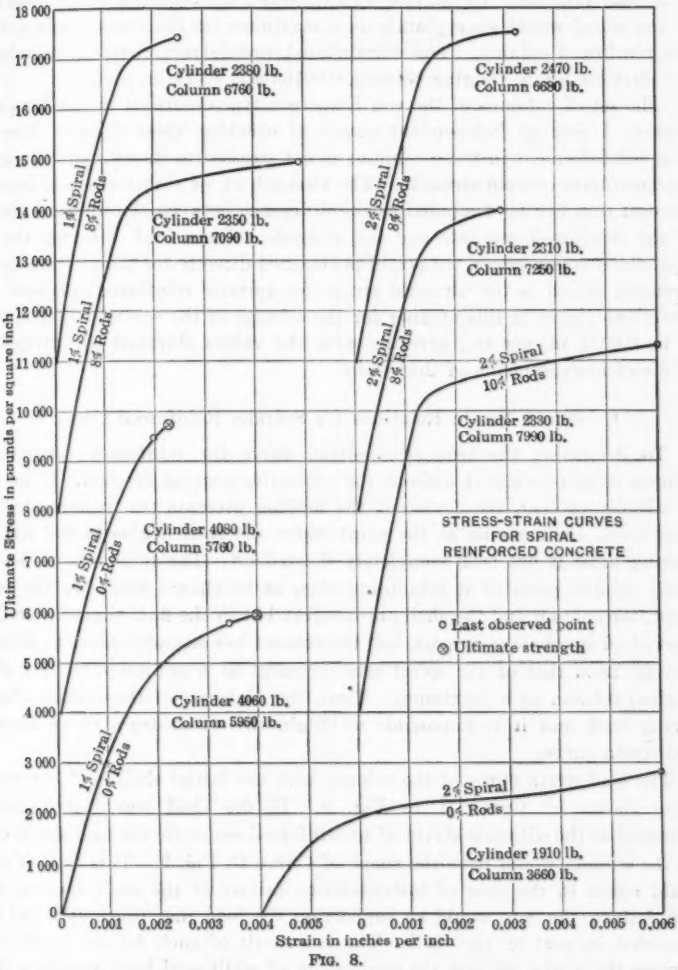


FIG. 8.

stress point in the stress-strain curve was the stressing of the spiral to the yield point. Referring to Equation (39), it will be noted that the second term in $\frac{r_s}{m} p_s$ would remain practically constant, as $\frac{1}{m}$ is not capable of any great increase, which it would have to undergo noticeably to increase the whole

term, and p_s would remain constant, as the yield point had been reached. It was shown that $e_r E_c$ increases considerably beyond the value at the ultimate stress of unreinforced concrete; yet for the part of the stress-strain curve at the stage where the stress is practically constant for increasing strain, $e_r E_c$ may decrease. In the case of a decrease, the reaching of the yield point of the spiral would show plainly as a maximum on the stress-strain curve of the reinforced column. The extrapolated stress-strain curves give values of the ultimate strain ranging between 0.00165 and 0.015 in. per in.

The set of columns of the von Emperger type described in the last part of Section I give an independent means of checking these figures. The cast-iron reinforcement in these columns is not stressed to its ultimate strength at the maximum column strength. The strength of, or rather the load borne by, the cast iron can be computed as the difference between the ultimate strength of the reinforced concrete and the ultimate column load. Having the cast-iron, stress-strain curve, the strain is obtained directly for the ultimate column strength, which is the ultimate strain for spirally reinforced concrete. The strain calculated in this manner for the average of the several columns proves to be 0.0034 in. per in., agreeing with the values obtained by extrapolation of direct measurements of the strain.

C.—Stress-Strain Relations for Spirally Reinforced Concrete

To determine the true stress-strain curve for reinforced concrete, the column should be tested without the protective coating or shell. If the shell is initially present, the course of the loading presents two phases which are interfluent, commencing at the point where the shell begins to fail and continuing until it has been completely sheared off. The total loss of the shell rarely occurs, parts of it remaining even at maximum load. In the initial phase, the column and the shell are integral, but in the final stage the shell has sheared off in whole or in part, and the column has an indeterminate diameter, varying from that of the spiral reinforcement as a minimum to that of the original column as a maximum. Thus, the diameter of the column changes during load, and it is impossible to obtain the stress-strain curve from the load-strain curve.

The load-strain curve of the column with the initial shell is of the general shape shown by Curve *A* in Fig. 9. If the shell was instantaneously destroyed at the ultimate strain of unreinforced concrete, the load-strain curve for the column would have the shape of Curve *C*, Fig. 9. This type of curve would result in the case of instantaneous failure of the shell, because after such failure, the core would be supporting the load that previously had been supported in part by the shell. The net result of such failure would be to increase the strain without the application of additional load, resulting in an action similar to that of steel at the yield point. From the load-strain curves obtained from tests on columns of this type, which do not show any indication of the marked changes of direction, it is proved that the shell fails gradually.

If a column without a shell is tested, a curve of the general form of Curve *B*, Fig. 9, will result, the break in the curve being more sudden, and with no

such sweep as that shown in the region, *a* to *b*, Curve *A*, Fig. 9. This wide bend of the stress-strain curve has been seen as an indication of the weakness of this type of column. The stress-strain curve for the core of the column, however, shows a much straighter curve to what might be termed the yield point.

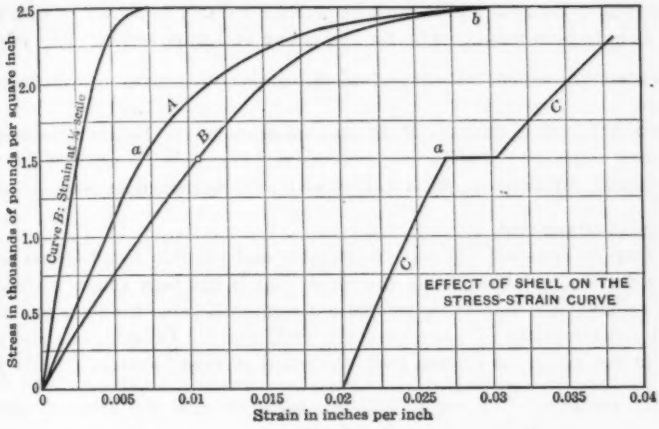


FIG. 9.

D.—Lateral Strains

Extrapolation of measurements by Professors Withey and Talbot give values of the ultimate lateral strain for spirally reinforced concrete, that lie between the limits of 0.0006 and 0.0030 in. per in. The greatest measured lateral strain is well before the point, *b*, Fig. 10, or the end of the yield-point elongation for steel wire. The lateral strains for this type of column to the failure of the concrete core are, of necessity, equal to the strains in the reinforcing wire. This shows that in every case the spiral was stressed at the most to yield point, and, frequently, much less, when column failure or maximum load occurred. The failure of the column is due to the failure of the concrete, and not to the failure of the spirals.

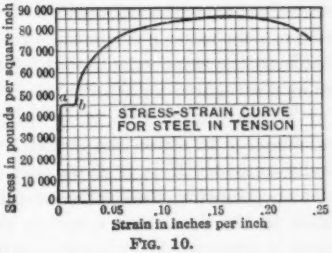


FIG. 10.

E.—Method of Failure of the Spirally Reinforced Column

From Equation (40) and a representative column stress-strain curve, the values of E_c for strains beyond the ultimate strain of unreinforced concrete can be calculated with fair accuracy. The unknowns in the equation are $\frac{1}{m}$ and E_c. All the other quantities in the equation, either directly or in-

directly, have been measured with accuracy sufficient for the purpose. The value that $\frac{1}{m}$ assumes at the ultimate strain of unreinforced concrete must be between the limits of 0.25 and 0.50. It is theoretically impossible for $\frac{1}{m}$ to be greater than 0.5 and it must be as large as 0.25, as shown by such readings as have been made and by the indication of future action. Any value of $\frac{1}{m}$ within this range, substituted in Equation (40), gives nearly the same value for E_c, the effect on E_c of the variation of $\frac{1}{m}$ between these limits being small. Therefore, E_c is determined with fair accuracy and $\frac{1}{m}$ within a range of 50 per cent.

Using in Equation (9) any conceivable and possible simultaneous set of values for the variables within the range that it has been shown possible for them to have, it is impossible to obtain a computed value for the stress in the spiral reinforcement of more than the yield point. Observations also show that, at the maximum column load, the spiral at most is stressed to the yield point. Assuming $\frac{1}{m}$ equal to 0.5, the value to give the greatest value to p_s, and the values of E_c, or $e_r E_c$, as obtained by Equation (40), it is found that, for a longitudinal strain of 0.01, Equation (41) gives a much lower value to p_s than was determined by Equation (40), which strain is known to be true. Substituting 0.01 as the value of the longitudinal strain, and every measured or possible value for the remainder of the variables in Equation (9), always leads to the same result, that is, a calculated value of p_s much less than the known measured value of p_s or its value calculated from Equation (40).

This is proof that it is impossible for a material having any rigidity whatsoever to stress the spiral beyond the yield point, no matter what that value may be, if it is more than a minimum of 35 000 lb. per sq. in. It is known, however, that compression of reinforced concrete columns eventually ends in the breaking of one or several of the spirals. This means that the load is transmitted through a concrete that has properties different from the original whole concrete. The change in the concrete may be accounted for in two ways, as follows:

a.—The concrete becomes a perfect fluid, transmitting the longitudinal compressive stresses directly to the spirals. If this hypothesis was true, the stress-strain curve of the column would show a great drop in the load at the point at which the concrete became fluid. Observations do not show such a drop, thus proving that this hypothesis is incorrect and that the concrete does not become a fluid.

b.—The concrete of the column shears or crumbles, shifting the load more directly on the spirals. The friction between the sheared or crumbled surfaces, together with the tension stresses in the spiral, would explain the high column

stress after the concrete had ceased to be intact. Such shearing would result in local stressing of the spirals in the shear plane. The breaking of several turns of the spirals at points directly under each other bears out the hypothesis. This is the only possible explanation of the failure of spirally reinforced columns. It is to be noted that the breaking of the column spirals in tension is only an inconsequential anti-climax and is without importance.

Observations on the tests of the von Emperger columns showed that the spirals failed after the maximum load had been applied, the cast iron breaking finally. Cast iron fails at a strain of about 0.01 in. per in. To cause failure of the spiral, a strain of 0.25 in. per in. is necessary. For a perfect fluid the longitudinal strain necessary to produce a strain in the spiral of 0.25 would be 0.5. As concrete, even at the ultimate spirally reinforced strain, is not quite a perfect fluid in the limiting case, the longitudinal strain necessary to cause the failure of spiral in tension would be at least 0.5. The ultimate strain, however, of cast iron, which is considerably greater than that of spirally reinforced concrete, is only 0.01 in. per in.

That the failure of the spirals in a column is due to an initial shear failure in the concrete is borne out by the fact that test columns after failure show a general shear and crumbling in the plane of the tension break in the spirals. At all points throughout the axis of the test column, the concrete was strained beyond the ultimate of unreinforced concrete, although, after testing the concrete column, the concrete will be sound, except where the spirals have failed.

The two columns, described in *Bulletin No. 466* of the University of Wisconsin, of strong concrete (about 4 900 lb. per sq. in.), show clearly by their stress-strain diagrams that failure occurred in the concrete, stress-strain measurements being made to maximum load. These curves are almost straight lines, and show no inclination to change direction before failure like the columns of lower strength, similarly reinforced. The stress computed from the measured strain of the spiral gives only about 12 000 lb. per sq. in. for a load just under the maximum. It appears impossible that the stress-strain curve should suddenly change its direction at right angles, or nearly so, just prior to failure. This would have to be the case if the spirals were the cause of the failure. Professor Withey concludes from observations that the failure of these columns was due to the failure of the concrete.

Modern compression testing machines apply load by means of increasing strains. Consequently, in a test of a column to destruction, the course of the phenomena is altogether different in some phases from that which would occur if the load was applied directly by means of weights. One great difference is in the time. It has been shown that in the spirally reinforced column the concrete fails first and shifts the load on the spirals. The means of increasing the load in the testing machine is the application of more strain, which is an exceedingly slow process, especially for the very large strains necessary to cause breaking of the spiral in a column of the type under consideration. A period of perhaps 15 min. would be required to accomplish in the testing machine what would occur instantly under a gravity load. The load would

be shifted to the spirals the instant the concrete failed, the column shortening rapidly, the spirals failing, and the column, as a whole, collapsing.

5.—Summary

1.—The increase in the strength of concrete due to spiral reinforcement is independent of:

(*a*) The yield point of the spiral when that point lies within the limits of 38 000 and 115 000 lb. per sq. in.

(*b*) The ultimate strength of the spiral when this value is between 60 000 and 150 000 lb. per sq. in.

2.—At maximum column load, the spiral is stressed to some value between limits of about 18 000 and 90 000 lb. per sq. in., or to the yield-point stress if it is less than 90 000 lb. per sq. in.

3.—The failure of the spirally reinforced concrete column is caused by failure in the concrete itself, by crushing, shearing, or a combination of both. As a corollary, the failure of the spirals in tension is not related to the column failure, but is a phenomenon beyond this point.

4.—The increased strength of the column is due to the increased strength of the concrete itself, because of the lateral restraining action of the spiral on it.

5.—There is an upper limit to the strength of the concrete that can have its strength increased by the use of spiral reinforcement. Beyond about 4 000 lb. per sq. in., the increase in strength due to spiral reinforcement diminishes rapidly with that of the unreinforced concrete, until at about 4 900 lb. per sq. in., the strength of the concrete is not appreciably increased by the addition of spiral reinforcement. Theoretical (by Equation (40)), and also actual, measurements show the increase in the strength of 4 900-lb. concrete to be approximately 5 per cent.

6.—Concrete has its strength increased (for a column of nine diameters) by spiral reinforcement, as given by the formula:

$$f'_{os} = f'_c\,(1 + 0.573\,r_s)$$

in which,

f'_{os} = the ultimate strength of the reinforced concrete;
f'_c = the ultimate strength of the unreinforced concrete; and,
r_s = the ratio of spiral reinforcement.

For a column of any length, the strength of the reinforced concrete is:

$$\frac{P_m}{A} = 1.27\left(1.00 + 0.573\,r_s\right)\left(1.000 - 0.0173\,\frac{L}{D}\right)f'_c$$

in which $\frac{L}{D}$ is the ratio of length to diameter.

7.—Longitudinal reinforcing rods add definite strength to the column, approximately 32 000 lb. per sq. in. of rod cross-section, the value depending on the length of the column. The strength including that added by the rods is given by the formula:

$$\frac{P_m}{A} = 1.20\left[(1 - r_r)(1.00 + 0.573\,r_s)\,f'_c + 36\,500\,r_r\right]\left[1.00 - 0.0183\,\frac{L}{D}\right]$$

SECTION III.—RELIABILITY

The knowledge of the reliability of the physical properties of a material is indispensable in engineering design, being equally as important as the knowledge of the average values of such properties. The most important of these properties are ultimate stress, yield-point stress, or other salient property of the stress-strain relationship, hardness, resilience, etc. The only properties of the material or structural element discussed in this paper are those of strength, namely, elastic limit, yield point, ultimate stress, and other possible salient features of the stress-strain relationship, endurance limit (stress), continued load stress, resilience, etc. Other physical properties, such as the resistance to wear, are unimportant in this consideration.

The reliability not only of the properties of materials, but also of built-up units is of the utmost importance, as that of the property of the final structure or structural element is the object sought. The reliability of the property of the material is of importance only because of its influence on that of the property of the structural element into which the material enters.

The allowable, safe working stresses, or the factors of safety, are surely functions of the reliability of some one of the strength functions of a material or structural element, and should be deduced with a proper consideration of the effect of this reliability. Committees appointed to determine the safe and allowable working stresses always give great weight to the reliability, or more properly, lack of reliability, of the strength of materials, especially for such materials as cast iron and concrete, in which the reliability is very low. If not explicitly considered, it is unconsciously taken into account with the instinctive knowledge which is, or should always be, possessed by those whose duty it is to draw up and revise the specifications for working stresses. Nevertheless, the reliability is never given the consideration it deserves. Even when cognizance is taken of the reliability and an attempt is made to allow for the lack of it, such attempt is made in the crudest manner. Objection will be made to a certain stress on the ground of the unreliability of the material, which results in a reduction in the allowed working stress to provide for the poor reliability. Although accurate results sometimes are obtained by thus using the judgment and intuition of those long familiar with the materials, this method is equally as likely to result in huge errors. Wherever possible it is being supplanted by the correct method which omits the personal equation and permits the results to be obtained mathematically from measurements made by any of a skilled number of experimenters.

Technical engineering literature contains frequent reference to reliability, but always in the vaguest manner; it is never expressed numerically or even in definite terms that would afford a means of comparison between the reliabilities of the strengths of two materials. That the strength reliability of steel or fabricated steel structural elements is high, and that the strength reliability of cast iron and concrete is low, is a true and obvious statement. It is, however, capable of limited application, especially in the determination of the influence of the reliability on the working stresses to be permitted. What

of the relative reliability of two steels of different composition and heat treatment, or of two materials that gave strengths for test specimens of 100, 140, 150, 160, and 200 for five specimens of the one and strengths of 110, 120, 150 180, 190 for five test specimens of the other, the average of both being the same? Although reliability may readily be intuitively perceived in a general sense, yet when refinements enter, the determination by intuition can readily be seen to fail completely.

How is the strength reliability of materials exhibited? Is cast iron an unreliable material because of the suddenness and violence of its failure? The Quebec Bridge was built of the most reliable material known, yet, between the first indication of failure and complete failure, so little warning was given that it was valueless either to prevent the failure or to enable any of those on the bridge to escape. It is true that this was an unfinished structure, but had failure of the finished structure occurred, due to innumerable possible causes, it would have been just as sudden. Spiral and rod reinforced concrete is regarded as having more strength reliability than unreinforced concrete. Yet, subsequently, it is shown that spiral and rod reinforced concrete and unreinforced concrete have practically the same reliability. Owing to the careless methods of fabricating the concrete and also in checking the strength of the concrete entering the structure, its strength is not always what it should be. For this reason, the longitudinally reinforced concrete may be regarded as more reliable, as only a part of the strength of the column, namely, that due to the concrete, is lowered by the concrete of inferior strength. The testing of spiral and rod reinforced concrete requires a comparatively long time for failure to be effected, even after the first positive signs. The long time necessary to cause failure is the result of the method of application of the load in the modern testing machine. Although the spirally reinforced column possesses much greater resilience and, therefore, can withstand shock much better, the static load that would cause the spirally reinforced column to fail would cause that failure to occur instantly. Evidently, then, the suddenness and violence of the failure or the length of time required for failure to occur in the testing machine is no indication of the strength reliability of the material or structural element.

Reliability is a measure of uniformity, and, therefore, can be measured by means of the variations of the individual specimens from the mean value of the magnitude of whatever property of the particular object that is under consideration. The process of determining the reliability for a specific case theoretically would necessitate the discovery of the law of the manner of the variations. Although research to determine the manner of variation is to be expected, such knowledge is not imperative. The variations may safely be assumed to be identical with those given by the standard error function. From observations of the variations it is now possible to compute, by the theory of probabilities, the value of the property under consideration, which will be exceeded in all cases. This may be stated as the value which will be exceeded only in all but an infinitesimal number of cases. To illustrate the

application, if a material has a mean strength of 2 000 lb. per sq. in., and a reliability number of 0.75, then every specimen made of this material would safely stand a stress of $0.75 \times 2\,000 = 1\,500$ lb. per sq. in. A more detailed discussion and development of the theory is given subsequently.

1.—Physical Measurements

Whether the desired result is a simple measurement of length, or the temperature of the sun, no two observers will arrive at the same measurement of the desired quantity, nor will the same observer be able to obtain in succession the same numerical result.

There is no such quantity as the "true" value, but it may be taken as that value of the quantity determined by the most refined means and instruments possible. Bach, Pearson, and others have proved that all things are relative, therefore, any measurement is relative, being a comparison of the object with a standard. The property of the object measured did not exist—the only reality was the measurement or comparison. The property cannot be separated from the object.

In general, the variations in results are due to possible differences in method, the personal equation, which enters perceptibly when the element of time is operative or a distance must be estimated, etc., difference due to errors in the instruments used, to uncorrected effects of light, temperature, electric and magnetic fields, atmospheric influences, all of which, in most cases, cannot be eliminated, except in the most refined experiments, and also those possible disturbing factors which as yet are unknown.

By making a large number of observations, using several methods, and having observations made by several observers and obtaining the average, obviously, a result much more accurate will be obtained than any single observation in the group. The errors or variation of any result from the true value will as likely be positive as negative, therefore, the sum of a large number of errors is so nearly zero that the difference can be neglected. Therefore, the averaging of a large number of observations automatically eliminates the errors, which is expressed by the statement that the errors are compensating.

In testing concrete, no attempt is made to correct for atmospheric effects other than to keep the air saturated with moisture during the first week after making the specimen, and to keep the specimen at approximately room temperature. The strength of concrete will be affected by the various errors in the machine in which the concrete is tested, and by the method of mixing it. Accidental eccentricity of loading and rate of application of load will cause appreciable variation in the apparent strength. The thoroughness of mixing, percentage of water used, variation in the average size and grading, and the strength of the coarse aggregate, etc., all tend to vary the strength of concrete made from aggregates of identical proportions.

The variations in strength of individual test specimens of any material, or of any composite or built-up structural element, nearly coincide in their mathematical relationship with the errors of observations as given by the well-known "error function" formula. The variation in strength of such a material as

rolled steel is not as apparent as it is in materials such as cast iron and concrete. This strength variation may be viewed as the result of the effort of the internal components, either microscopic or macroscopic, or of the magnitude of the aggregate of concrete, to arrange themselves according to some average scheme; the change in arrangement (the arrangement being produced during fabrication) would cause differences in strength.

The formula describing the manner in which errors occur, as theoretically developed a century ago, has been substantiated by repeated experiment for the errors of observation and like cases on which the original hypotheses for developing the formula were based. Experiment also shows that the variation in strength of individual test specimens from the mean or average value of the strengths of those test specimens, follows, as nearly as can be expected from the large magnitude of the errors and the relatively small number of specimens tested, the general equation. The so-called "error function", which mathematically describes the possibility and magnitude of any error is a function of the form:

$$y = \frac{h}{\sqrt{\pi}} e^{-h^2 v^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (43)$$

in which,

v = the magnitude of the error;
y = the relative probability of the error, x, occurring;
h = a constant, also the intercept or the relative probability of the error being zero.

e and π have the usual significance.

By custom:

$$\frac{h}{\sqrt{\pi}} \int_{-\infty}^{+\infty} e^{-h^2 v^2} \, d\, v = 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (44)$$

or, expressed otherwise, the total probability is arbitrarily established as unity.

Plotting a large series of observations, the abscissas representing the error or variation of the individual specimen or reading from the mean or average value of the observed quantity, and the ordinates the number of observations that were made having the same error equal in magnitude to the value of the ordinate, a curve of the same general contour as that given in Equation (44) is obtained. By a change in the constant to make the integral, $\int_{-\infty}^{\infty} y \, d\, v = 1$, the theoretical curve is nearly obtained, the experimental curve for various reasons not coinciding exactly with the theoretical one. The necessity for an enormous number of observations in order closely to approximate the theoretical curve is usually the main reason for the lack of coincidence.

The value of the constant, h, can also be algebraically determined from the formula:

$$h = \sqrt{\frac{n}{2 \Sigma v^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (45)$$

in which,

n = the number of observations; and

v = the variation of the individual observation from the mean of the observations.

The variation in the ultimate strength of a material obeys or follows the laws stated, with only a slight general exception. The standard probability curve extends to infinity in the positive and negative directions. That the strength of a material can be less than zero, however, is absurd. In Fig. 11, if D is the average strength of a material, and the strengths of the specimens and the percentage number of specimens are the abscissas and ordinates, respectively, the curve will be the same as the probability curve between D and infinity, but will be a different function between O and D, of the form shown in the diagram in exaggeration, and is such that,

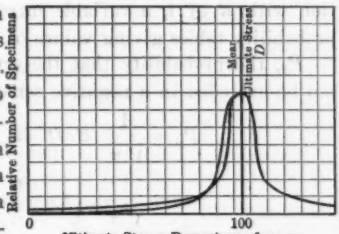


FIG. 11.

$$\int_O^D y\, d\, v = \frac{1}{2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (46)$$

The difference between the results obtained from Equation (46) and Equation (44) is small, so that the general or standard formula and probability tables may be used.

The probable error is defined as the value that is as likely to be greater than the real error of the result as it is to be smaller. This can also be expressed: The probability of the error of the result being less than the probable error is equal to the probability of the error of the result being greater than the probable error.

The probable error is determined by the equation:

$$R \equiv p\,.\,e\,.\,= 0.6745 \sqrt{\frac{\Sigma\, v^2}{n\,(n-1)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (47)$$

for the mean of a number of observations; and,

$$R \equiv p\,.\,e\,.\,= 0.6745 \sqrt{\frac{\Sigma\, v^2}{n-1}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (48)$$

for a single observation. That is, Equation (47) gives the probable error of the arithmetic mean of several observations, and Equation (48) gives the probable error for any one observation, which may be one of the observations made in determining Equation (47).

The probable error, R, although an inverse measure of the reliability, is capable of no application in the present consideration, and further consideration of the properties of the error function is necessary. The fundamental equations are:

$$R \equiv 0.6745 \sqrt{\frac{\Sigma\, v^2}{n-1}} = \frac{0.4769}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (49)$$

$$t = h\, v_0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (50)$$

from which is derived,

$$t = 0.04769\,\frac{v_0}{R}, \text{ or } v_0 = \frac{t\,R}{0.04769} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (51)$$

in which v_0 is any value of the error or variation connected by a 1:1 relation with R by t, a variable introduced to simplify probability calculations, probability tables giving values of t for any desired probability.

P_t, the probability that the error, v_0, corresponding to the value, t, will not be exceeded in a negative sense, is given by the equation:

$$P_t = \frac{1}{\sqrt{\pi}}\,Erf\,t = \frac{1}{\sqrt{\pi}}\int_0^t e^{-t^2}\,dt$$

$$= \frac{1}{\sqrt{\pi}} \cdot \left[t - \frac{t^3}{3} + \frac{1}{2\,!} \cdot \frac{t^5}{5} - \frac{1}{3\,!} \cdot \frac{t^7}{7} + \cdots\right] \quad \ldots\ldots\ldots\ldots (52)$$

It can be readily proved that:

$$[1 - P_t] = \frac{1}{\sqrt{\pi}}\,Erfct = \frac{1}{\sqrt{\pi}}\int_t^\infty e^{-t^2}\,dt$$

$$= \frac{1}{\sqrt{\pi}} \cdot \frac{e^{-t^2}}{2\,t} \cdot \left[1 - \frac{1}{2\,t^2} + \frac{1.3}{2^2\,t^4} - \frac{1.3.5}{2^3\,t^6} + \cdots\right] \quad \ldots\ldots\ldots (53)$$

which is the probability that the error, v_0, will be exceeded.

Values of t can be determined for any assigned value of P_t or $(1 - P_t)$, by means of Equations (52) and (53). From t determined by Equations (52) and (53), can be found the value of v_0, the error whose probability of being exceeded is $(1 - P_t)$.

If x_m is the mean value of the variable, and x_0 the value of the variable which is v_0 in error,

$$x_0 = x_m - v_0 = x_m - 2.10\,t\,R$$

$$x_0 = x_m \left[1.00 - \frac{2.10\,t\,0.6745}{x_m}\sqrt{\frac{\Sigma\,v^2}{n-1}}\right]$$

$$= x_m \left[1.000 - 1.415\,t\sqrt{\frac{\Sigma\frac{v^2}{x_m^2}}{n-1}}\right]$$

$$= x_m \left[1.000 - 0.01415\,t\sqrt{\frac{\Sigma\frac{v^2 \cdot 100^2}{x_m^2}}{n-1}}\right]$$

$$= x_m \left[1.000 - 0.01415\,t\sqrt{\frac{\Sigma\,v_p^2}{n-1}}\right]$$

as $v_p = v$, expressed in percentage of $x_m = \frac{v}{x_m} \times 100$.

It is now necessary to determine the value of t so that the possibility of v_0 being exceeded is infinitely small. The factor, $1.000 - 0.01415\,t\sqrt{\frac{\Sigma\,v_p^2}{n-1}}$, for this value of t will be termed, N_r, the reliability number. N_r can also be

defined as the ratio of the value of the variable which will always be exceeded to the mean value of the variable.

Fig. 12 gives the graph of $1 - P_t$ and t within the range of values necessary in this analysis. Mathematically, t should have the value of infinity to give $1 - P_t$ the value of zero, but the selection of the value of t for use depends on the final application of N_r, the reliability number, this application being the determination of the safe working stress. It is known that there are certain causes producing the variation in the variable the N_r of which is to be determined. If this variable is the ultimate strength of concrete or reinforced concrete, severe loss of strength due to these causes greatly aggravated, such as voids, draining of the fine aggregate and cement to leave only the coarse aggregate in place, will result in an appearance of the finished product, that will cause it to be rejected. It is not necessary, therefore, to assume too minute a probability. The minuteness of the probability of the strength or value of the variable being inferior to that given by the substitution of the assumed value of t need be only relatively minute. To illustrate the impossibility of assuring a safe value for N_r, no mathematical equation can assure that, in a unique case, all the cement may not be omitted from the mix, or a substance incapable of producing any strength whatever be substituted. These possible but very improbable accidents give a concrete of zero strength, yet their occurrence can be made to be without ultimate danger by taking test specimens from every batch of concrete.

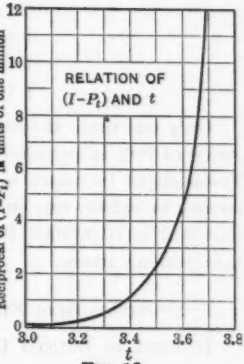


FIG. 12.

In the present consideration, no direct conception of the relative magnitude of the order of millions, or of the necessity for relatively absolute safety, is possible. Some comparison or other means must be used to bring the problem within the grasp of understanding. Such a comparison is available in the safety from death of the normal man. Calculations show that the chances that a human being will die within 10 min. of any predetermined instant is 1 in 10 000 000. This probability will be accepted as perfect safety and is synonymous with infinity. The problem may also be regarded as the determination of the value of the reciprocal of the probability $(1 - P_t)$, which is infinity. A similar problem is that of the number of repetitions of stress that may be considered as infinite, or which will be sufficient to cover the number of repetitions received by a component material in the life of a machine. Values of infinity have been suggested* for several machine and structural elements which vary greatly with use. For steam turbine shafts, the given value is 15 000 000 000, for railroad bridge chords it is 2 000 000.

The probability of 0.00001% will be almost surely accepted as being completely satisfying. Suppose, however, that this probability is assumed as un-

* *Mechanical Engineering*, 1919, p. 731.

necessarily small, that a comparatively larger probability value, say, 0.0001% (1 chance in 1 000 000), or 1 chance in 500 000, is sufficient. Comparing the values of t (with the reciprocal of which the working stresses are roughly proportional), for these two probabilities, there is:

Probability:

$1 - P_t$	t.
1 in 500 000	3.31
1 in 1 000 000	3.39
1 in 10 000 000	3.68

Only one-tenth the safety is obtained by a decrease of t of 8½%; by an increase of 10% in permissible strength only one-twentieth the safety is obtained. These slight increases in working stresses, with the accompanying large decreases in safety, can only lead to the conclusion that the value of 0.00001%, that is, 1 in 10 000 000, for the probability should be chosen as the basis of the safe working stress.

2.—Strength of Steel and Determination of Reliability Number

It must be noticed that the variation in strength developed by several specimens is not necessarily a measure of the reliability of the composite element or material in question. When a factor enters, the variation of which causes variation in the strength of the material and there is previous knowledge of the effect of this factor, or ability to control it, then the variation in strength due to the factor is not due to any unreliability of the material itself, and can be separately determined and discounted.

Take the example of steel in tension: The variation in magnitude of the carbon content of steel causes the strength to vary greatly (other constituents and treatment remaining constant). Low-carbon steel has an ultimate strength of about 55 000 lb. per sq. in. High-carbon steel can be manufactured with an ultimate tensile strength of 150 000 lb. per sq. in. If no consideration is given to the carbon content of a group of wires selected at random and tested, the logical conclusion, on the determination of the strength reliability, would be that steel wire is very unreliable. However, if the carbon content is noted, one perceives that the strength is a function of the percentage of carbon in the steel, and by arranging the wires into groups having the same carbon content, it can be seen that, for any group, the strength is very uniform. On determining the reliability number for each group, it is seen that steel is a very reliable material. The treatment that steel receives in fabrication also varies the strength, but for similar carbon content, usually the same treatment is given in the case of reinforcing wire. This discloses the importance of determining, as far as possible, any factors that cause variation in strength, both to eliminate weaknesses that they cause and to eliminate their effect on the apparent reliability.

Table 10 gives the reliability number, N_r, for the various types of columns analyzed in Section I, and also the product of N_r and the ultimate strength, which will be the maximum safe stress of one application of load.

Casual inspection of Table 10 will show what experience has proved in a broad way; steel stands first in reliability, being far superior in this respect to concrete or cast iron. The group of columns depending on concrete for their main strength show close agreement in their reliability numbers, which mutually shows that the premises on which the formulas in Section I are based, must be correct, and the variation in strength of the column is due to the variation in strength of the concrete, and, therefore, concrete is not made less reliable by the introduction of reinforcement.

TABLE 10.

	r	$0.0772r$	N_r	Product of N_r and ultimate stress.
Structural steel	2.05	0.1559	0.84	*29 400
von Emperger type	2.36	0.182	0.82	†8 200
Structural steel and concrete	4.31	0.332	0.67	‡1 660
Concrete and rods	5.61	0.432	0.57	1 420
Concrete and spiral	5.63	0.434	0.57	§
Concrete rods and spiral	7.00	0.540	0.460	§
Concrete unreinforced	5.83	0.450	0.550	1 370
Cast iron	12.00	0.927	0.073	‖5 100

* Net strength steel in compression equals yield-point stress, or about 35 000 lb. per sq. in.

† The von Emperger type will be more fully considered subsequently.

‡ Ultimate stress of concrete $\times$ N_r for concrete of 2 500 lb. ultimate strength.

§ Varies greatly with percentage of reinforcement and concrete strength.

‖ Ultimate strength = 70 000 lb. per sq. in.

The great variation in strength of the cast-iron specimens should not be finally indicative of this type of column; it can be explained partly by the fact that the cast-iron specimens tested were regular commercial specimens, as against the carefully prepared laboratory specimens of concrete, and, also, that they were made in various foundries with different grades of iron, the result being a variation in strength due to the use of different materials similar to that caused by different carbon content in steel, previously discussed. This latter affect could have been eliminated by basing the strength of cast iron on small test specimens from the same pour; but, unfortunately, these small specimens were not made. With the advances that have been made in the technique of casting iron columns, there is no need to place all cast-iron columns in the same unreliable category.

The reliability number has been determined from the ultimate strengths for the group of test columns. The determination of that number for the columns of a new structure, however, would necessitate the testing to destruction of a number of the group, which is an impossible commercial procedure. Test cylinders of the concrete composing the columns must form the basis for the determination of the reliability number for the column. The variation in strength of concrete in test cylinders and in a column of unreinforced concrete is not identical. To obtain the N_r for the reinforced column from concrete test cylinders, the ratio of the N_r of test cylinders to columns (reinforced) must be introduced as a factor.

SECTION IV.—ALLOWABLE WORKING STRESSES OR FACTORS OF SAFETY

1.—Introduction

Methods of mathematical analysis of the stresses and strains in any conceivable type of structure have already been sufficiently developed so that, for any theoretical structure (perfect in dimensions, composed of homogeneous materials having straight-line stress-strain relations, etc.), the stresses and strains at every point in it for any possible application of load can be determined. The hypotheses, that plane sections remain plane in bending, that E, the modulus of elasticity, is constant, assumptions of general application, and those used in special cases, as for dams, that horizontal sections move frictionless on each other, etc., by means of which calculation of the stresses and strains are made possible or greatly simplified, are, in most part, only close approximations, frequently giving results of the same magnitude. Their use, however, always gives magnitudes for the variables, that will not be exceeded at the specific point in the structure by the actual value of the variable.

Contrasting with the complete development of the methods of mathematical analysis of the stresses and strains in a structure are the crude methods used in the determination of the stresses which may be safely withstood by materials or composite structural elements, or the determination of the variations of the actual stresses from the theoretical for each point in the material or structural element composing the structure. As is usual with an undeveloped branch of investigation, so in the determination of the working stresses to be allowed, extraneous factors are included as essentials in the consideration, no systematic procedure has been developed, and the whole subject is vague and indefinite, clouded with engineering superstition, such as the belief that repetition of stress (fatigue) causes crystallization in ferrous metals.

The present allowable working stresses are based primarily on experience. If the original stress was too great, the resulting failures, in the course of time, cause increase in the factor. It seldom happened that the stress was made too small, the common occurrence being an allowance much too large, which is slowly reduced to the point where minimum strength with safety is found. Undoubtedly, the experience method eventually produces good results; but it is rather dangerous and is exceedingly slow, tedious, and costly in operation. Without the testing machine, the experience, or trial-and-error, method is the only one possible. There is no longer an excuse for using this antiquated method. With the wealth of new materials, old materials made by new and refined processes, and as a check on the strength to be allowed those materials the strength of which constantly varies with the output, there is urgent need for a short method of determining definitely the allowable working stresses directly from the results of tests made in the common forms of testing machines.

2.—Definition of the Problem

The strength of the structure and the loads that it will carry is a function of the stresses in the members and the strength of the material or element of the structure in which these stresses are developed. The design of the safe structure necessitates the knowledge of the maximum stress that the material at any designated point or individual structural element can carry with safety, and, also, the knowledge of the maximum stress that the structure, due to all the variations and imperfections of the actual structure and its component materials and elements from the theoretical and perfect, together would impose on that material or individual structural element. The loading which would cause the maximum safe stress at any point in the structure, would then be the maximum permissible loading on the structure, and, as a corollary, the maximum permissible loading would cause, at most, the safe working stress at any point in the structure.

The determination of safe working stresses, therefore, does not involve details apart from the material or structural element the safe stress of which is desired. The safe stress of concrete in compression is independent of everything except the ability of that concrete to resist an infinite number of applications of stress of the magnitude to be designated the safe working stress.

The problem of ascertaining the maximum stress that can be developed in each particular member of a structure by the application of load, is one of design independent of the consideration of safe stresses, and should be so considered. The structure should be designed so that with any and all possible imperfections, mishaps, etc., the maximum permissible or designated loading on it will cause, at most, a stress equal to the safe working stress.

Increase in the factor of safety, a result of including too many and extraneous factors, does not give increased safety. Human nature is such that increase in the factor of safety leads to increased risks in loading and much heavier over-loading, with the result that the total imposed load will bear even a larger ratio to the ultimate strength of the structure for the larger factor of safety. The smaller the factor of safety the safer a structure becomes, as an overload of 100% for a factor of 2 is judged more accurately than an overload of 900% for a factor of 10. Consequently, all influences tending to increase the factor of safety should be eliminated, if possible.

3.—Previous Analyses of the Problem

Professor Talbot* contends that the working stresses and working factors in structures are influenced by:

1.—Lack of uniformity of material, variation in material and its fabrication.
2.—Increased deflection due to: (*a*) repeated load; and (*b*) time load.
3.—Uneven distribution in load among members.
4.—Unconsidered stresses due to settling, unevenness in material, etc.

* *Bulletin No. 10*, Univ. of Illinois, Eng. Experiment Station, Paragraph 24.

Professor Talbot also gives the following as important in the problem:

"(a) the stress actually brought upon a member of a structure by an assumed loading may be materially higher than the assumed working stress, and (b) the stress actually developed in the member may be much higher comparatively (i. e., with respect to its own ultimate strength) than even this increased amount would indicate. * * * What point should be fixed upon as the basic point, upon which a working factor covering uneven distribution of load, uncertainty of quality, effect of repeated loading, etc., may be based, will depend upon the nature of the material and the conditions of the structure. * * *"

And, in the summary,

"10.—* * * as a basic point [that is, a basic stress in the determination of the allowable working stress] * * * the choice of a value corresponding to a deformation equal to one-half of the deformation at point of failure is suggested. This [stress] by the parabolic relation, is equal to three-fourths of the ultimate strength. Having selected a basic point, a working factor will then be chosen to cover contingencies and emergencies, * * * methods of fabrication, nature of the load, manner of application, etc."

This whole determination of the problem may be censured because of indefiniteness and the inclusion of too many independent factors, many of which can be treated separately.

As a base stress on which to derive the working stress, the figure, three-fourths of the ultimate stress, can have no other basis than intuition. It has been shown in Section III that, to allow for the usual variation in the strength of concrete, one-half the ultimate strength must be used to insure safety. Professor Talbot leaves us about where we started when he says that this basic stress must be corrected for contingencies and emergencies, nature of load, etc. The problem has only been begun at this point. The usual method of solving it is to keep in mind all the factors influencing the problem, to meditate gravely the question and, then, announce with the conviction of infallible intuition, that the factor should be 4. This method may be convincing in metaphysical speculation, but it is much out of place in the solution of engineering problems.

Professor Talbot's items may be separated into:

A.—Those quantities which influence the stress actually produced in any particular member of the structure: Uneven distribution of load among members, unconsidered stresses due to settling, unevenness in the material, contingencies and emergencies unforeseen, methods of fabrication, etc.

B.—Those factors that influence the actual (safe) stress to be permitted in the material or structural element as in the structure. These factors are:

1.—Lack of uniformity of material, variation in material and in its fabrication.

2.—Increased deflection (preferably strain) due to: (*a*) repeated loads; and (*b*) time loads.

3.—Uncertainty of quality, effect of repeated and time loading, all of which solely influence the ultimate strength or safe stress which is to be imparted to the material or structural element.

Unevenness in material has been placed under both Items *A* and *B*. It must be considered twice, as it causes variation in the stress actually in a given member, and, also, in the ultimate strength of that member. Variation in strength of one member may cause a variation in stress in some other member.

As previously stated, the items referred to in Item *A* are problems of design and should not be investigated in consideration of the working stress. A unit volume of concrete has the same ultimate strength no matter where it is placed, and it should be permitted an invariable working stress. Its position in the structure does not change its safe stress.

The net result of the lack of uniformity and variation in material may be measured with precision by the reliability number, as demonstrated in Section III, and allowance made for the effect of the variation in strength. The variation in fabrication can be dealt with in two ways, each of which will give decisive results. The first and best method is to take test specimens on the work as a check on the workmanship and leave no variable for this quantity. There will be, therefore, no need for introducing a factor into the allowable working stresses to provide for the variation. The second method is to determine the ratio of the strength of the specimens made in the laboratory to those made in the field. This ratio will then give the field strength of a certain aggregate, if laboratory specimens are made from it. An excellent series of such tests is described subsequently, which give valuable results. It can now be seen that the "uncertainty of quality" and "variation in material and in its fabrication" can be eliminated.

The three items of uncertainty and variation mentioned have, in effect, only one net result, namely, the variation in strength of the material in the structure. This may be due to uncertainty of material, an ambiguous term, to variation in strength due to fabrication, and in differences in erection, such as the treatment during the preparation of the concrete, or the erection of the structural steel. The variation in strength due to fabrication can be eliminated, as the material to be used can be tested and its strength and other properties determined before it is incorporated in the finished structure. In design, a certain fabricated strength is specified, and the material as delivered to the work must possess this strength as a minimum. The variations in strength of the material in the finished structure—the most important item—has been discussed in Section III and will be reviewed subsequently. The increased deflections (more properly, the increased strains), and the decreased ultimate strength due to repeated and time loading, are of vital importance.

Messrs. Turneaure and Maurer* give the following items governing the selection of the allowable working stresses:

(*a*) Variation and imperfection in material and workmanship.
(*b*) Uncalculated stresses, such as secondary stresses due to unequal settlement, and, usually, those due to temperature change.

* "Principles of Reinforced Concrete Construction," Second Edition, Chapter V, p. 209. No mention of the subject is made in subsequent editions, the authors evidently leaving the matter in the hands of committees of the proper engineering society.

(*c*) Dynamic effect of live load, if not provided for by an allowance for impact.

(*d*) Possible increase in live load over that assumed, or rare applications of excessive loads.

(*e*) Deterioration of the structure.

These and similar items are important only in proportion to their application. No further use of them than their enumeration makes this mention superfluous. In this instance, the statement of the items seems to have been sufficient, for later in the book, without any further reference or application of them in the intervening pages, the allowable working stresses for a concrete column reinforced with vertical rods is determined by "taking" an ultimate stress that can "reasonably be expected" for such a column, and dividing this by an arbitrarily chosen "factor of safety of 4". The use of the phrase, "reasonably be expected", and the arbitrary selection of 4 as the factor of safety are hardly in keeping with a branch of knowledge with such a scientific basis as engineering. These methods are not based on any processes that can be logically followed by different individuals to obtain identical results. Had some other authority given the factor, it might have been different. It is true that the factor of safety for reinforced concrete is taken by the best engineers and, in general, accepted as about 4. The Joint Committee on Concrete and Reinforced Concrete in its 1917 Report, suggests 22½ per cent. The difference in using a factor of 3 instead of 4 results in a saving of 25% of material. Consensus of opinion would have it that a factor of 3 is too low; yet how, by this method of intuition, can it be disproved that 4 is too high and that 5 is the lowest factor to be used? Can this vital matter be decided by the opinion of the majority? Failures in concrete structures are of too frequent occurrence. On this basis alone, the factor of safety should be increased; yet, it is well known that all these failures are due to inferior concrete, and instead of increasing the factor of safety, a rigid control of the making and placing of concrete and an insistence on a high strength should be strongly urged.

Item (*a*), variation and imperfection in material and workmanship, is a factor vitally affecting the allowable working stress. Although it has long been recognized that there are variations in the strength of materials and structural elements, especially pronounced in concrete, due to "variation and imperfection in material and workmanship", no method has been used to determine even crudely the effect of such variation and imperfection on the strength. The influence of imperfection and distortion of the structure, such as that indicated in Item (*b*), is a consideration of design and in no way affects the working stress. There must be some limit to the factors that must be recognized and met in the figure set for the safe working stress. To illustrate the reason for this necessary limitation by an exaggerated case, no factor of safety could ever compensate for the omission of all the cement from a batch of concrete. It is possible to determine with reasonable accuracy the stress that a material or structural element will safely carry. If part of the problem is capable of solution, why introduce unnecessary, unknown, and problematical factors such as emergencies, possible overloads,

unequal loading, eccentric loading, settlement, etc. These factors vary with the individual structure, the foundation, and the use to which the structure is placed. The emergencies and overloads should be the utmost conceivable for the particular case. The design is the proper place to take cognizance of all these factors. The problem of settlement should be solved by the use of foundation areas proportional to the load on them, and the structure so designed, possibly by a change from the present monolithic construction, in order that unequal settlement will not cause secondary stresses or unequal distribution of load directly or indirectly due to it.

The dynamic effect, as in Item (*c*), should always be considered. Working stresses cannot be determined from two independent criteria. There is no way to allow for a rare application of an excessive load, except to design so that such a load produces, as a maximum, the working stresses in the individual members. The working load would then be a percentage of this rare excessive load. The phenomenon of fatigue reduction of strength permits a few applications of 100% overload, but this is an inherent property of materials, and cannot be adjusted.

Item (*e*), deterioration of the structure, is ambiguous. It can be taken to mean reduction in the ultimate strength due to time loading, to loss of strength in time independent of the load, or to reduction in strength due to loss in area of the structural elements by abrasion, disintegration, or other influences.

It has been determined experimentally that concrete does not lose any material part of its maximum strength with age. The effect of time loading has been determined definitely. For the particular case, the quantity of material lost by abrasion during the life of the structure must be predetermined and extra material added to the structural element. Disintegration should be prevented, or if that is impossible, allowance for it should be made. Such effects as abrasion, disintegration, etc., cannot be considered, being factors of design and unrelated to determinations of safe stress.

4.—A Rational Scheme for the Determination of Safe Working Stresses

It is not necessary to consider internal column reactions of the nature of the initial strain and correlated stress on reinforcing rods in longitudinally reinforced concrete columns due to shrinkage of the concrete, as these phenomena occurred in the test columns. The results showing a stress of 31 000 lb. per sq. in. in the reinforcing rods of longitudinally reinforced columns possibly indicate that initial compression due to concrete shrinkage was present, as claimed by several authorities. Neither is it necessary to take into consideration the method of failure of the column or compound structural element. It is immaterial, for example, whether the rods of a longitudinally reinforced concrete column cause the failure of the column or whether the failure is independent of the action of the rods. The safe working stresses can be based on formulas determined mathematically from the results of column tests, eliminating all speculation on the internal behavior of the column and its elements.

The safe working stresses for a type of structural element must be a function of the following factors:

1.—The maximum stress that can be applied in infinite repetition, or for one sustained application for an infinite period (time load), without causing (*a*) a permanent set, or (*b*) a strain at the last repetition or end of the infinite time load, either being greater than some maximum permissible strain value determined by factors inherent in the structure itself or otherwise determined, such as, by the presence of plastered coating on the walls. For an elevated railway structure, for example, a working strain of 0.0035 in. per in. in the supporting members would not be excessive.

2.—The formula, empirical or exact, that has been confirmed experimentally, expressing in terms of all the component variables the strength of the structural element.

3.—The reliability number, N_r, based on the variation in strength of the individual specimens from the strength indicated by the formula given in Factor (2).

4.—The relation between the strengths of the specimens prepared in the laboratory (such as are always used in tests) and the strength capable of being developed by the material as commercially fabricated. When check test specimens of the concrete as mixed on the work are taken, this factor is unnecessary.

The safe working stress, or preferably the safe working load, reduces to a simple equation of the type:

$$P = N_r L_e F(V. M f'_c) A \quad \ldots\ldots\ldots\ldots\ldots\ldots (54)$$

in which

P = the safe column load;
N_r = the reliability number;
L_e = the endurance limit or maximum stress possible on account of strain or set consideration;
$F(V. M f'_c)$ = the mean strength of the column empirically determined; a function of the variables: (*a*) strength and percentage concrete; (*b*) percentage steel reinforcement; and (*c*) slenderness ratio, etc.
f'_c = the ultimate stress of concrete test cylinders;
M = a factor equal to unity when check test specimens are taken of the concrete, and, approximately, 0.6 when they are not;
A = the cross-sectional (active) area of the column (equal to the area within the spiral diameter in that type of column).

A.—Essential Data on Concrete

Fatigue.—J. L. Van Ornum, M. Am. Soc. C. E., has found,* by means of large numbers of compression test cylinders and also test beams reinforced so as to fail in compression in the concrete, that the endurance limit for concrete is 50% of its ultimate strength, proper methods being used to eliminate the effect of variation in strength in the concrete of the individual test specimens. The curve illustrating the fall in strength with the number of repetitions is given in Fig. 13.

* *Transactions*, Am. Soc. C. E., Vol. LI (1903), p. 443, and Vol. LVIII (1907), p. 294.

The strains resulting from repeated applications of load must also be obtained. Professor Van Ornum has done this, obtaining the modulus of elasticity, E_c, for each additional application of stress throughout the cycle. He found for stresses of more than 50% of the ultimate stress, that each application of load decreased E_c, the greater the stress ratio to the ultimate stress, the more rapid the decrease, with the final result that the concrete failed in compression. He also found for the endurance limit stress (50% of the ultimate), or greater, that E_c decreases, but reaches an asymptotic value. For an initial value of $E_c = 3\,500\,000$, and an application of the endurance limit stress, the asymptotic value is $E_c = 2\,250\,000$, 65% of the initial value. For this loading, the permanent set is quickly achieved and does not increase. This property of concrete is of great value. From stress considerations, the endurance limit is the maximum stress value that can be used; from purely strain considerations, the endurance limit may also be used.

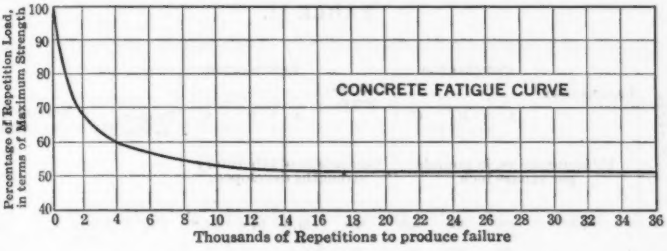


FIG. 13.

B.—The Relative Strength of Laboratory Made and Commercially Made Concrete

The correct methods of making and preserving concrete to develop maximum strength, prevent deterioration, etc., have been developed and perfected by laboratory research. Commercial concrete cannot be expected to be made with all the necessary refinements and precautions, because of the high inherent cost and the impossibility of placing the strongest concrete properly in complicated reinforcement. There is no reason, however, to tolerate the present general product that is furnished.

The results of a series of tests given in *Technological Paper No. 58*, of the U. S. Bureau of Standards, will be used as data in this discussion of working stresses. The results are given of tests on the strength developed by a definitely proportioned dry aggregate as mixed and made into concrete test cylinders by several contractors and on similar specimens made by the Bureau. The data are not exactly those desired in this instance, as the contractors wanted to make the strongest possible concrete instead of the usual run, the cylindrical test specimens permitting them to make a stronger concrete, the aggregate and proportions being correctly chosen. If left to the contractors, the resulting aggregate would cause a further reduction in strength. The results of these tests are given in full in Table 11. It will be seen that the average strength developed by the Bureau was 66⅔% higher than that developed

by the contractors. This difference was due to variation in thoroughness in mixing, placing, and, chiefly, the too great content of water. In the fabrication of difficult reinforced shapes, it would be almost impossible to use the quantity of water necessary to develop maximum strength for a given aggregate. Aside from the special conditions where it is impossible to use the quantity of water to develop the maximum strength, flagrant abuses such as a large surplus of water, insufficient mixing, incorrect proportions, incomplete tamping, etc., will not tend to furnish a concrete as strong as that produced in the laboratory. It would be insufficient to have all technical men thoroughly familiar with the principles of making concrete. The average workman will do his work in the easiest possible manner, and although he will do it to conform with the specifications in the presence of the inspecting engineer, he will lapse in his absence. The only positive check on the quality of the concrete is the test cylinder.

TABLE 11.

Age at test, in weeks.	Laboratory.	Contractor.			Laboratory.	Contractor.			Proportions.	Mixture and aggregate.
		A	B	C		A	B	C		
	Ultimate stress, in pounds per square inch.				Percentage of laboratory strengths developed.					
4	700	880	450	580	100	125.6	64.3	82.8	1:3:6	Hand and machine mixed, gravel.
	.....	610	560	400	....	.....	....	....	.....	.....
	1 010	850	560	670	100	84.0	55.3	66.3	1:3:6	Machine mixed, gravel.
	950	420	400	690	100	44.2	42.2	72.7	1:3:6	Machine mixed, limestone.
	3 050	1 250	1 620	2 210	100	35.8	46.3	68.2	1:2:4	Hand and machine-mixed, gravel.
	2 310	1 450	1 790	1 820	100	62.7	77.5	78.8	1:2:4	Hand and machine-mixed, gravel.
Mean.	.....	.....	.....	.....	100	70.5	57.1	72.7	66.8	Mean of three means.
13	1 100	880	590	690	100	80.0	53.7	62.8	.....	.....
	1 610	1 260	950	1 120	100	78.2	59.0	69.5	.....	.....
	1 210	740	550	850	100	61.2	45.5	70.3	.....	.....
	3 150	1 750	2 160	2 120	100	55.6	68.6	70.5	.....	.....
	3 320	1 900	2 980	2 830	100	57.3	90.0	85.5	.....	.....
Mean.	.....	.....	.....	.....	100	50.5	52.6	59.2	54.1	Mean of three means.

5.—Working Stresses

An abstract of the working stress specifications for reinforced concrete columns for the principal American cities, and calculations of the stresses based on them for all large cities in the United States, has been published by Mr. Pierce P. Furber.*

The Specifications recommended in 1917 and the Tentative Specifications submitted in 1921, by the Joint Committees are to be accepted as more nearly

* "A Summary of the Column Code", *Journal*, Am. Concrete, Inst., 1916, p. 181.

the standard to which in time the various codes will be revised, and have been chosen as the basis of the general criticism.

The 1917 Specifications* for "Columns" are, as follows:

"(*a*) Columns with longitudinal reinforcement to the extent of not less than 1% and not more than 4%, and with lateral ties of not less than ¼ in. in diameter, 12 in. apart, nor more than 16 diameters of the longitudinal bar: the unit stress recommended for axial compression, on concrete piers having a length of not more than four diameters, in Chapter VIII, Section 3. [This stress for the concrete piers is 22½% of the expected strength of the concrete.]

"(*b*) Columns reinforced with not less than 1% and not more than 4% of longitudinal bars and with circular hoops or spirals not less than 1% of the volume of the concrete and as hereinafter specified; a unit stress 55% higher than given for (*a*), provided the ratio of unsupported length of column to diameter of the hooped core is not more than 10."

Chapter II of the 1917 Specifications† on "Design and Supervision", contains the following:

"(*d*) Inspection * * *:

"5. Strength of the concrete [to be obtained] by tests of standard test pieces made on the work."

The 1921 Tentative Specifications‡ are, in substance, as follows: The safe load on spiral reinforced columns is given by the equation:

$$P = A_c f_c + n f_c p A \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (A)$$

in which,

A = the area of the concrete core enclosed within the spiral;

P = the total safe axial load on a column whose $\frac{h}{R}$ is less than 40 (R is the radius of gyration of the cross-section of the column within the spiral, and h is the column height);

p = the ratio of effective area of longitudinal reinforcement to area of concrete core;

$A_c = A\ (1 - p)$ = the net area of concrete core;

$n = \frac{E_s}{E_c}$ (for which the load is not stated);

f_c = the safe load on the unit of core cross-section and is given by the equation:

$$f_c = 300 + (0.10 + 4\ p)\ f'_c$$

f'_c = the crushing strength of concrete in 6 by 12-in., or 8 by 16-in., test cylinders.

"The longitudinal reinforcement shall consist of at least six bars of minimum diameter of ½ in., and its effective cross-sectional area shall not be less than 1% nor more than 5% of the enclosed core. * * * The spiral reinforcement shall not be less in amount than one-fourth the volume of the longitudinal reinforcement. * * * Reinforcement shall be protected everywhere by a covering of concrete cast monolithic with the core, which shall have a minimum thickness of 1½ in. in square columns, and 2 in. in round or octagonal columns."

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1133.

† *Loc. cit.*, p. 1111.

‡ *Proceedings*, Am. Soc. C. E., August, 1921, Papers and Discussions, p. 102.

Section 67* of the 1921 Specifications states that the fire-proofing shall be "not less than 2 in. in beams, girders and columns" where the aggregate is limestone, or its fire-resisting equivalent, or 3 in. for quartz.

The safe load for longitudinally reinforced columns is given by the equation:

$$P = A'_c f_c + A_s n f_c \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (B)$$

in which,

A'_c = the net area of concrete in the column (total column area less steel area);

A_s = the effective cross-sectional area of the longitudinal reinforcement;

f_c = $< 0.20 f'_c$, and is the safe load on the unit of column cross-section;

f'_c = the crushing strength of the concrete in 6 by 12-in., or 8 by 16-in., test cylinders.

The longitudinal reinforcement must not be greater than 2% nor less than 0.5 per cent.

For both types of columns with $\frac{h}{R}$ larger than 40, the safe load is given by the equation:

$$P' = P\left(1.33 - \frac{h}{120\,R}\right)\dagger \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (C)$$

in which, P corresponds to values given by Equations (A) and (B) and P' is the value of the safe load for the column of the given value of h and R.

In Table 4,‡ Section 2 of the 1921 Specifications, it is stated that:

"If the proportions to be used in the work are selected from the table without preliminary tests of the materials, and control tests are not made during the work, the mixtures in bold-face type shall be used."

A.—Columns Reinforced with Vertical Rods Only

The analysis in Section I shows that:

1.—The rods at ultimate column strength are stressed approximately to their yield point.

2.—The yield-point strength in the reinforcing rods was developed independently of the amount of the reinforcement, a few of the test columns having as much as 4% of rods. The strength developed by the larger percentage is capable of much closer check because of the far greater influence on the column strength than a relatively small percentage, such as 1 per cent.

3.—The rods do not decrease the strength developed by the concrete in any individual column, and the concrete does not decrease the strength of the rods. The contention is untrue that, in the process of destruction of the column, the rods buckle and cause the surrounding outer layer of concrete to scale, thus starting and aiding the failure of the column. The phenomenon is

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* *Proceedings*, Am. Soc. C. E., August, 1921, Papers and Discussions, p. 80.

† Equivalent to $P' = P\,(1.33 - 0.033\,\frac{L}{D})$.

‡ *Proceedings*, Am. Soc. C. E., August, 1921, Papers and Discussions, p. 111.

a phase in the failure of the testing of this type of column, and represents the buckling of the rods after the maximum strength of the column and concrete has been passed and the concrete has broken, allowing the rods to buckle outward.

4.—The strength of the column varies considerably within the limits of the lengths of columns tested and permitted in practice.

5.—To develop the full strength in the reinforcing rods, it is necessary that the ultimate strain of the concrete reinforced shall be, as a minimum, the strain necessary to develop the full yield-point stress in the steel reinforcement, which is 0.00125 in. per in. The minimum ultimate stress of concrete to develop this minimum ultimate strain is 2 000 lb. per sq. in.

6.—The reliability of the strength of the rods and of the concrete developed in the rod reinforced columns is independent of the amount of the rod reinforcement.

Besides these five factors governing the safe working stresses to be permitted and of which no cognizance is taken in the present and tentative code, there is another important factor, namely, no credit as regards strength is given the protective shell of the spirally reinforced column, yet there is no concomitant reduction made from the actual concrete cross-sectional area of the longitudinally reinforced column.

If the spirally reinforced column requires fire protection, the rod reinforced column does also. Fire will destroy the surface concrete of the rod reinforced column and decrease the strength-bearing area in the same manner as in the spirally reinforced column; and, as is proper, the strength developed by the shell of the spirally reinforced column is not considered. A shell of equal thickness should be used as fire protection in the rod reinforced column, and no strength credit given to this layer of concrete. That the shell of the spirally reinforced column is destroyed at ultimate column strength is a valid reason for not allowing credit for it in calculations of safe working stress. Heretofore, however, the ultimate strength of the spirally reinforced column has never been the basis of the calculations for working stress as the ultimate strain is so great, the ultimate strain of unreinforced concrete being the basis for the determination of the allowable working stresses. The protective shell adds strength to the column at the ultimate strain of unreinforced concrete, exactly as the outer layer of concrete adds strength to the rod reinforced column. If this layer is given no credit in the spirally reinforced column, due to its function as fire protection, then certainly a layer of equal thickness should be allowed for fire protection in the rod reinforced column, and given no strength credit.

Development of the Formula.—The formula giving the column strength relative to the necessary variables, as developed in Section I, is:

$$\frac{P_m}{A} = 1.28\left(r_c f'_c + 31\,000\, r_r\right)\left(1.00 - 0.0265\,\frac{L}{D}\right) \qquad (55)$$

It is now necessary to correct Equation (55) by the reliability number, N_r, to allow for the natural variation in strength, and by the factor, L_e (= 0.5),

to allow for repeated and time loads, that is, to obtain the endurance limit. Equation (55), corrected by these factors, then becomes:

$$\frac{P}{A} = 1.28\ L_e\ N_r\left(r_c\ f'_c + 31\,000\ r_r\right)\left(1.00 - 0.0265\ \frac{L}{D}\right) \ldots\ldots (56)$$

in which,

P = the safe column load;
A = the effective column area (that is, the column area less the outer layer of concrete designated as fire protection);
r_c = the ratio of concrete in the column $(1.00 - r_r)$;
r_r = the ratio of longitudinal reinforcing rods;
f'_c = the average ultimate strength of the concrete as used in the column and determined by test cylinders;
L = the length of the column;
D = the diameter of the effective concrete area in the same units as the length of the column;
N_r = the reliability number; and
L_e = the endurance limit for concrete (0.5).

For design, N_r is either the value that may reasonably be expected from a given aggregate and proportions, or that may be determined by test specimens. In the actual structure, $N_r f_c$, as determined by test specimens, must be, as a minimum, the value assumed for design.

For the rod reinforced column, analyzed in Section I, $N_r = 0.57$, the figure used in the safe stress computation in this Section.

Equation (56) now becomes:

$$\frac{P}{A} = 0.365\left(1.00 - 0.0265\ \frac{L}{D}\right)\left(r_c\ f'_c + 31\,000\ r_r\right) \ldots\ldots\ldots (57)$$

An alternate procedure, not to be recommended, is to be used when no check test specimens are taken of the concrete as mixed on the job. Although haphazard, this method is possibly the only plausible one in small work and in cases when those in charge cannot be convinced of the necessity of test specimens. The alternate method of determining the permissible stresses is to incorporate a factor of 0.6 in Equation (57) to correct for the strength of the concrete as determined by test specimens made from laboratory mixture.

Equation (57) would then read:

$$\frac{P}{A} = 0.365\left(1.00 - 0.0265\ \frac{L}{D}\right)\left(0.6\ r_c\ f'_c + 31\,000\ r_r\right) \ldots\ldots\ldots (58)$$

The constant, 0.6, was obtained from *Technological Paper No. 58* of the U. S. Bureau of Standards, and is the ratio of the strength of concrete mixture by skilled laboratory workers to that obtained by employees of contractors. Although the concrete as mixed on the work would not in every case be only 60% of the strength of concrete mixed in the laboratory, yet, if no test specimens are taken, the capable makers of concrete must be penalized equally with the makers of ordinary concrete.

Table 12 has been compiled to demonstrate the great variation in the safe working stresses for columns of extreme dimensions, and also the great

difference between those computed on the strength of concrete, as usually obtained, and the present allowable working stresses.

TABLE 12.—SAFE WORKING STRESS TABULATION FOR ROD REINFORCED COLUMNS.

A	B	C		D	E	F	G	H
Concrete, ultimate strength, in pounds per square inch.	Ratio of length to diameter, $\frac{L}{D}$	PERCENTAGE OF REINFORCEMENT.		PERMISSIBLE WORKING STRESSES, IN POUNDS PER SQUARE INCH BY		CALCULATED SAFE STRESSES, IN POUNDS PER SQUARE INCH BY		Working stress in concrete, in pounds per square inch, for stress indicated in Column F.**
		Spiral.	Rods.	1917 Specifications.	1921 Specifications.	Equation (57).	Equation (58)¶.	
1 500	6	0	0.50†	....	321	505	322	425
			1.00‡	338	342	552	370	
			2.00§	338	384	642	461	
			4.00‖	338	....	823	647	
			10.00	....	....	1 365	1 200	
1 500	12*	0	0.50	....	300	410	269	425
			1.00	338	319	447	300	
			2.00	338	359	520	375	
			4.00	338	....	666	525	
			10.00	....	....	1 110	975	
1 500	15	0	0.50	....	268	362	220	425
			1.00	....	285	395	264	
			2.00	....	321	490	330	
			4.00	....	....	588	462	
			10.00	....	....	975	855	
3 000	6	0	0.50	....	633	960	597	850
			1.00	675	666	1 006	642	
			2.00	675	732	1 092	730	
			4.00	675	....	1 265	911	
			10.00	....	....	1 780	1 450	
3 000	12*	0	0.50	....	597	747	465	850
			1.00	675	630	783	498	
			2.00	675	602	850	570	
			4.00	675	....	984	710	
			10.00	....	....	1 382	1 125	
3 000	15	0	0.50	....	528	686	436	850
			1.00	....	556	720	458	
			2.00	....	611	780	522	
			4.00	....	....	905	651	
			10.00	....	....	1 270	1 035	

* Maximum length permitted by 1917 Specifications.

† Minimum permissible under 1921 Specifications.

‡ Minimum permissible under 1917 Specifications.

§ Maximum permissible under 1921 Specifications.

‖ Maximum permissible under 1917 Specifications.

¶ Based on actual concrete of 60% of the strength of laboratory specimens.

** Actual stress in the concrete of the column under working load. Column F gives average stresses in both steel and concrete.

Criticism of the 1917 Specifications.—

1.—The allowable working stresses bear very little relation to the safe stresses, as determined by the writer.

2.—Based on the strength of concrete to be expected from a mix of a certain nominal strength, the working stress permitted varies as much as 40% greater than the safe stress as determined by the writer.

3.—If concrete is well made and check specimens are taken, much higher (as much as 100%) safe stresses can be sustained than are permitted by the 1917 Specifications.

4.—The permissible working stress should vary with the percentage of reinforcement and the length of the column.

5.—The maximum percentage of permissible reinforcement is not large enough. Tests show that as much as 4% of reinforcement develops the same stress *per se* as the least percentage of reinforcement.

6.—A thickness of shell equal to that demanded for spirally reinforced columns as fire protection should be required in longitudinally reinforced columns, no strength allowance being given to the shell.

Criticism of the 1921 Tentative Specifications.—

1.—The allowable stresses disagree with the safe calculated stresses, although not as widely as those in the 1917 Specifications.

2.—The 1921 Tentative Specifications are an improvement on the 1917 Specifications in that they permit (*a*) longer columns, and (*b*) varying working stress, within the limits.

3.—The 1921 Tentative Specifications have less merit than the 1917 Specifications in that they decrease the already too small maximum permissible percentage of longitudinal reinforcing rods.

4.—The working stresses, in all cases, are lower than the safe calculated stresses, provided the concrete in the column is equal to the nominal concrete strength on which design has been based, and for many choices of mix and percentage reinforcement are lower than the safe calculated stress by as much as 25 per cent.

5.—For the strengths to be expected from a nominal strength concrete, the permissible stresses of the 1921 Tentative Specifications are much above the safe calculated stresses for many possible selections of variables.

Comment on Stress Values Determined in This Paper.—

1.—The unit working stress in the concrete of the column as determined in this discussion is not excessive, being only 28.4% of the ultimate strength.

2.—With this conservative stress from the old general viewpoint, much greater total column loads are obtained.

3.—The great increase in strength due to increase in reinforcement is demonstrated.

B.—Columns Reinforced with Longitudinal Rods and Spiral

The analyses in Sections I and II show that:

1.—The increase in strength of concrete due to spiral reinforcement is proportional to the strength of the concrete and to the percentage of spiral reinforcement to a maximum of 2%, there being no experimental data for a larger percentage of reinforcement.

2.—The increase in strength of the concrete due to the spiral reinforcement is independent of the ultimate strength of the spiral when this value lies between the limits of 60 000 and 150 000 lb. per sq. in.

3.—Longitudinal reinforcing rods in a spirally reinforced column contribute approximately 36 500 lb. per sq. in., or the yield-point stress per unit area of rod, to the column strength at maximum load, the data being available only for a maximum rod reinforcement of 10 per cent.

4.—To the ultimate strain of approximately 0.0015 in. per in., the protective shell is undestroyed and in great part is intact and serviceable as a fire protection at the ultimate column strain (maximum column load). At the strain of 0.0015 in. per in., the protective shell considerably increases the load supported by the column depending on the ratio of the area of the shell to that of the core. This increase is approximately 50%, and depends on the ratio of core to shell areas.

5.—The column strain at maximum load, or ultimate column strain, is approximately 0.0035 in. per in.

6.—The reliability of the ultimate or maximum strength of this type of column is only slightly less than for unreinforced concrete or for concrete reinforced with longitudinal rods.

7.—The strength reliability does not vary with: (*a*) the amount of spiral reinforcement with a maximum of 2%; (*b*) the amount of longitudinal rod reinforcement with a maximum of 10%; and (*c*) the column length with twelve column diameters as a maximum.

From an inspection of these qualities possessed by the spiral and rod reinforced column, it is at once apparent that the safe working stresses permitted should depend on the percentage of reinforcement, the length, and the relation of the other column variables. This is more clearly brought out in Table 12, of the calculated safe working stresses, which are shown to vary almost 100% for the limits of the variables permitted by the 1917 Specifications for the one permitted working stress.

Continued compression of specimens of the spiral and rod reinforced type of column, far beyond the ultimate and maximum stress and strain, has caused the general destruction of the shell, the spirals to be broken by tension, buckled longitudinal reinforcing rods, buckling of the column itself, and, in general, a complete destruction of the column proper. This has led to a general belief that these phenomena occur simultaneously with the point of maximum load, and a refusal to accept the ultimate column strength as the basis for the calculations of the safe working stresses. It is immaterial how a structural element fails. When failure occurs, the element is useless and, therefore, all other elements supported by it or transmitting loads to or from it are likewise useless in the system.

The ultimate column stress should be the basis of the calculations of the safe working stress. That some procedure has been adopted at some time or place is with many of far greater weight than the most logical theory, no matter how positive and incontrovertible the proof. Refer to the working stresses suggested by Considère and adopted by the French Commission, and given for the rod and spiral reinforced column by the formula:

$$f = f_c\,(1 + 15\,r_r + 32\,r_s)$$

in which,

f = the safe working stress;

f_c = the safe strength of the plain concrete, taken at 28% of the ultimate strength of the concrete in the form of cubes (equivalent to 18½% of the strength in the form of cylinders, according to the writer);

r_r = the ratio of longitudinal reinforcement;

r_s = the ratio of spiral reinforcement, with the criterion that the maximum working stress permitted shall be 0.6 the ultimate strength of the concrete.

In passing, it may be noted that the maximum stress permitted by Considère is about 73% greater than that permitted by the 1917 Specifications of the Joint Committee.

Returning to the development of the logical argument for the ultimate stress as the basis for the determination of the safe working stress: The arguments against such a determination are:

1.—The unreliability of the strength developed by the concrete.

2.—The column buckles.

3.—The column reaches maximum (ultimate) load at a very great strain, too great to permit in a structure.

4.—The marked yield point of the column.

These arguments may be answered categorically, as follows:

1.—From the results of the mathematical investigation in Section I, as previously summarized, the rod and spiral reinforced column is shown to be almost as reliable as the rodded column; the reinforcing elements contribute a definite part of the strength; and, finally, a mathematical equation being derivable, the ultimate strength reduced by the reliability number can be used accurately to determine the safe strength of the column, no matter how great the variations in strength may be.

2.—The column buckles after maximum load has been reached and passed. The steel column, generally recognized as perfection in structural work, also buckles after maximum load has been attained. The bend in the column in maximum load can be understood to be negligible from the consideration of the ultimate strain (that is, strain at maximum load). This strain at a maximum is 0.006 in. per in. Any bend in the column would manifest itself as an addition to this measure of compressive strain. Assuming the strain, 0.006, as due to the flexure of the column, a figure much too large, as the greater part of the measured compression is a compression strain in the column, the mid-point of a 10-ft. column would be laterally deflected about $1\frac{1}{2}$ in. This amount of deflection or bending is not a condition to be permitted permanently, but is not excessive at the ultimate strength. The working stresses are far below this stress, and this condition of bending and concomitant ultimate stress is, by the process of derivation, never for safety to be equalled.

3.—The great ultimate strain of spirally reinforced concrete is an advantage instead of a disadvantage. It permits of distribution of the load throughout the structure. The great ultimate strain is also an advantage, because of the concomitant great ultimate resilience. The column can absorb, just previous to failure, a large amount of energy in comparison to the rod reinforced column.

4.—The fallacy of the yield point of spirally reinforced columns has been brought out in Section II and may be summarized, as follows: The initial column consists of core and shell, integral and intact. With increase in strain and stress, at the ultimate strain of concrete, the shell is gradually destroyed, cracks, and falls from the column. The result is that the stress, previously

distributed over the whole cross-section, is finally distributed over the core area, resulting, for a small increase in load, in a great increase in stress and a natural relatively sudden increase in strain. Thus, the yield point really is that point at which greatly additional stress is impressed on the core without being visibly denoted on the testing machine.

There is one more important argument for the ultimate stress as a basis, that is, the advantage of the protective shell, which may be summarized, as follows:

1.—The shell, commencing to crack at the ultimate strain of unreinforced concrete, gives warning of overload and also of possible failure. Reference to Table 13 shows that the actual safe working stress in the concrete itself is about 32%, as computed by the method developed in this study. Overload sufficient to cause noticeable cracking would then indicate 200%, certainly a large leeway in load variation and also a sufficient warning of impending failure, under about 100% still greater load.

2.—The shell is sound and assumes about 50% of the column load for the ordinary column proportions for all stresses within the working range.

3.—The shell will be decreased in strength in part or in whole, according to the destroying influences of fire or overload beyond the unit ultimate stress of unreinforced concrete. The shell, therefore, is sound and a perfect fire protection at normal working loads and stresses, which will not be the case in the event of both these influences, but the probability of the two occurring simultaneously is extremely small. If the fire protection is perfect, there will be no overload in case of fire. Imperfect protection would possibly result in the collapse of upper floors, which would result in unequal distribution of the weight of such floors and the impact overload due to their fall.

Formula for Safe Working Stress.—Using the maximum or ultimate strength of the spiral and rod reinforced concrete column as the basic strength, the formula for the safe working stress in this type of column, derived as for the rod reinforced column, is:

$$\frac{P_m}{A} = 1.20\, N_r\, L_e \left[(1 - r_r)\,(1 + 0.573\, r_s)\, f'_c + 36\,500\, r_r\right] \times \left[1.00 - 0.0183\, \frac{L}{D}\right] \qquad (59)$$

in which,

P_m = the total safe load on the column;
A = the area of column cross-section, within the spiral;
r_r = the ratio of longitudinal reinforcing rods;
r_s = the ratio of spiral reinforcement;
f'_c = the average strength of the concrete, as tested in the form of small test cylinders;
L_e = the endurance limit, taken as 50% of the ultimate strength;
$\frac{L}{D}$ = the ratio of column length to diameter (of spiral); and
N_r = 0.46, as determined for the spiral and rod reinforced column in this paper.

TABLE 13.—WORKING STRESSES FOR SPIRAL AND ROD REINFORCED COLUMNS.

A	B	C		D	E	F	G	H	I	J
Concrete, ultimate strength, in pounds per square inch.	Ratio of column, core length to core diameter.	PERCENTAGE OF REINFORCEMENT.		PERMISSIBLE WORKING STRESSES, IN POUNDS PER SQUARE INCH.		CALCULATED SAFE STRESSES, IN POUNDS PER SQUARE INCH.		WORKING STRESS IN CONCRETE IN POUNDS PER SQUARE INCH, FOR STRESS INDICATED IN COLUMN F.		Permissible stress, in pounds per square inch,† spiral neglected, column of same dimensions as chosen in Column I.
		Rods.	Spiral.	1917 Specifications.	1921 Specifications.	Equation (59)	Equation (60)	Shell neglected.	Shell included.*	
1 500	6	1.00	0.25	.....	582	506	340	395	253	510
		5.00	1.25	.....	1 274	1 040	807	592	380	
		1.00	1.00	523	582	662	433	543	348	
		4.00	2.00	523	.....	1 115	812	740	474	
		10.00	2.00	.....	.....	1 605	1 320	740	474	
1 500	10	1.00	0.25	.....	582	466	311	395	253	510
		5.00	1.25	.....	1 274	955	740	592	380	
		1.00	1.00	523	582	608	397	543	348	
		4.00	2.00	523	.....	1 020	745	740	474	
		10.00	2.00	.....	.....	1 473	1 212	740	474	
1 500	15	1.00	0.25	.....	486	413	277	395	253	425
		5.00	1.25	.....	1 065	850	659	592	380	
		1.00	1.00	.....	486	540	353	543	348	
		4.00	2.00	.....	.....	912	663	740	474	
		10.00	2.00	.....	.....	1 310	1 078	740	474	
3 000	6	1.00	0.25	.....	800	923	590	790	506	1 000
		5.00	1.25	.....	1 248	1 650	1 167	1 184	760	
		1.00	1.00	1 026	800	1 232	775	1 080	696	
		4.00	2.00	1 026	1 167	1 870	1 268	1 480	950	
		10.00	2.00	.....	.....	2 315	1 750	1 480	950	
3 000	10	1.00	0.25	.....	800	848	541	790	506	1 000
		5.00	1.25	.....	1 248	1 510	713	1 184	760	
		1.00	1.00	1 026	800	1 130	1 072	1 080	696	
		4.00	2.00	1 026	1 167	1 715	1 162	1 480	950	
		10.00	2.00	.....	.....	2 120	1 602	1 480	950	
3 000	15	1.00	0.25	.....	668	758	481	790	506	853
		5.00	1.25	.....	1 073	1 345	954	1 184	760	
		1.00	1.00	.....	668	1 036	635	1 080	696	
		4.00	2.00	.....	974	1 529	1 035	1 480	950	
		10.00	2.00	.....	.....	1 890	1 426	1 480	950	

*Based on 16-in. core diameter, 20-in. outside shell diameter (2-in. shell required for fire-proofing).

†That is, the total permissible column load (by the 1921 Specifications) on the 20-in. diameter column divided by the cross-sectional area of the 16-in. diameter column.

Correcting for the lower strength of commercially fabricated concrete by the factor, 0.6, Equation (59) becomes:

$$\frac{P_m}{A} = 1.20\, N_r\, L_e \left[0.6\,(1 - r_r)(1 + 0.573\, r_s)\, f'_c + 36\,500\, r_r\right] \times \left[1.00 - 0.0183\,\frac{L}{D}\right] \quad \text{.....(60)}$$

$$1.20\, N_r\, L_e = 0.276$$

Table 13 gives permissible stresses by the 1917 and 1921 Specifications and safe stresses as computed by Equations (59) and (60) for comparison.

Criticism of the 1917 Specifications.—

1.—The working stresses permitted by the 1917 Specifications are in disagreement with the safe calculated stresses determined by Equation (60) (in certain limiting cases to a maximum of 33% greater), based on concrete of the strength commercially to be expected from that of a mix of a certain nominal strength.

2.—For concrete of actual strength equal to that selected in design, all the permissible stresses are less than the safe calculated stresses, but the ratio beween them varies greatly with the percentage of reinforcement.

3.—There is only one permissible stress for a wide range of reinforcement and variation in column length, whereas the safe calculated strength varies approximately 100% for the maximum permissible variation in this reinforcement.

4.—The percentage of spiral reinforcement should not be held between the narrow limits of 1 and 2. Although the upper limit of 2% is reasonable, as there are no available data on columns with greater percentages of reinforcement, the lower limit should be less than 1%, and, if limited at all, it should be governed by the minimum diameter of wire that can safely withstand without deformation the handling ordinarily given it.

5.—The allowable stresses may be more than 100% higher than the present allowable stresses for percentages of steel reinforcement larger than those permitted by the 1917 Specifications, but within the limits of reinforcement in test columns.

Criticism of the 1921 Tentative Specifications.—

1.—All the allowable stresses for concrete with an ultimate strength of 1500 lb. per sq. in. are slightly higher (15% and more) than the safe calculated stresses, where the actual strength of the concrete equals that of the concrete assumed in design.

2.—For the strength of concrete to be expected as commercially fabricated from a concrete mix of a nominal strength of 1500 lb. per sq. in., the permissible stresses are much higher (to a maximum of 75%) than the calculated safe stresses.

3.—Provided that the actual strength of the concrete of the columns is equal to the nominal strength of 3000 lb. per sq. in., the permissible working stresses

are less than the safe calculated stresses. The ratio between them is also nearly constant for the permissible range of reinforcement.

4.—For concrete of the strength to be commercially expected from a nominal 3 000 lb. per sq. in., all the permissible stresses are higher, to a maximum of 50%, than the safe calculated stresses.

5.—It is possible to select a range of percentage reinforcement so that the following absurdity occurs: By removing the spiral from a column, the allowable working load is higher by 25% in the example chosen, than that on the spirally reinforced column. The example is, as follows:

Column dimensions: Core, 16 in. in diameter; area, 201 sq. in.; outside diameter of column, 20 in. (a 2-in. shell is required by the specifications for all reinforcement).

Reinforcement: Spiral, 0.25%; rods, 1.00%.

Concrete strength: 3 000 lb. per sq. in.

Ratio: Length to diameter = 10.

The permissible load on this column, if it is considered as spirally reinforced, is 160 600 lb. If the spiral is not considered, the load is 201 600 lb. The removal of the spiral reinforcement will not increase the strength of the column or its safe working load.

6.—The upper limit of the permissible percentage of spiral reinforcement should be 2 instead of 1.25.

7.—No fixed ratio between the percentages of spiral and of rod reinforcement is necessary.

Aggregate Specifications.—It is appropriate to call attention to a suggested improvement in the specifications for aggregate. All criteria may show an aggregate to be defective, yet if the strengths of the concrete test cylinder are acceptable, the aggregate will be accepted. Mr. C. M. Chapman* gives suggestions for the strength of test specimens as the basis of acceptance in place of the tests of the various components. He summarizes the specifications, as follows: The materials used shall be of such quality and such proportions as to produce a concrete which shall show a compressive strength of 2 500 (or 2 000 or 1 500) lb. per sq. in. at the age of 28 days when tested in accordance with the standard methods of testing.

Argument for Present Stresses.—

1.—The actual stress produced in the concrete in a column by the safe working load, as derived in this paper, is, in the extreme case, a maximum of 31.5% of the ultimate concrete strength.

2.—By a process of logical reasoning, resulting in the present stresses for several selections of column variables, these stresses agree with those recommended by the 1917 or 1921 Specifications. If, for several cases, stresses can be calculated entirely from a formula agreeing with the magnitude of the safe stresses determined after twenty years of effort, it is proof of merit in the present method. Consistency, therefore, should make the permissible

* "A New Form of Specification for Concrete Aggregate," *Proceedings*, Am. Soc. for Testing Materials, Vol. XVI, Part II, p. 180.

stresses coincide, or at least bear a fixed ratio throughout, with the values obtained by the methods of this paper.

C.—Structural Steel Columns Embedded in Concrete

The strength of this type of column is a simple summation of the strengths of the two components, the steel and the concrete. For the working stress in the steel, in the present instance, the stresses now used are satisfactory, as it is the writer's purpose to discuss only concrete structures. The stress to be allowed in the concrete will be given by the equation:

$$f_0 = L_c N_r f'_c \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (61)$$

the variables having the usual significance. The variables have the values to give approximately,

$$f_0 = 0.3 f'_c$$

Thus, 30% of the strength of the concrete, as determined in test cylinders, is the safe working stress for the concrete in this type of structure.

D.—The von Emperger Type of Reinforced Column

The von Emperger type of column presents a case in which the application of the methods of determining the safe working stress, as followed in the previous types of columns, would lead to erroneous results. The apparent reliability of the von Emperger column would indicate a safe stress of a single short application of 83.4% of its average ultimate strength, which would mean approximately 31 500 lb. per sq. in. as the working stress in the cast-iron reinforcement. These values for the safe stress in the column and in the cast-iron reinforcement are, however, greatly in error, as will be shown.

The ultimate and maximum strength of the von Emperger column is the sum of two independent quantities: The strength of the spirally reinforced concrete part at the normal strength for concrete thus reinforced, and the strength of the cast iron at the ultimate strain of the reinforced concrete. It is possible that the surrounding concrete would cause the cast iron to assume a larger stress before failure, but until the matter has been definitely decided, the only possible action must be negative.

That the safe working stress cannot be as large as the apparent high reliability indicates, can be proved. The formula:

$$R = \sqrt{r_1^2 + r_2^2}$$

in which, r_1 and r_2 are probable errors of the same quantity due to different operating causes, and R is the actual and final error of the quantity, is a standard and accepted formula in the theory of errors. This formula can be applied in the present case, in which, R is the inverse measure of the strength reliability of the structural compound element, and r_1 and r_2 are similar quantities for the two different components of the compound element.

Let r_1 be the probable variation due to the cast iron and r_2 the probable variation due to the concrete. In the series of test columns, the variation in strength of the cast-iron reinforcement at ultimate column load was not a function of, or variation in, the strength of the cast iron; it depended on the ultimate strain of the reinforced concrete. Assuming that the ultimate strain

of the concrete is constant, r would also be constant, and this indicates that the strength of the cast iron is constant, which is absurd. The tests of the compound element give no measure of the variation in strength of the cast-iron reinforcement, and, therefore, no indication of the safe stress for it.

The safe working stress in the cast-iron reinforcement should not be greater than that used if this cast iron was a separate column without any surrounding concrete. The safe stress for cast iron, as calculated in this paper, is extremely low, about 3 000 lb. per sq. in. As previously stated, this low strength is derived from comparatively poor specimens, and it is probable that tests on vertically cast specimens, such as are now on the market, would indicate much higher safe stresses, even possibly exceeding the safe stress for steel.

Possibly, the embedding of the cast iron in concrete strengthens the cast iron by some stiffening action, preventing any individual specimen of low strength from breaking at a strength as low as it would, if it had not been embedded. To give credit for such increase in strength in the cast iron and permit higher working stresses, based on the ultimate strength of the column, a large number of columns would have to be tested, in order to determine whether a cast-iron core of low strength would reduce the strength of the column, and, if so, to allow for this reduction in the reliability number, N_r.

DISCUSSION

EDWARD GODFREY,* M. AM. SOC. C. E. (by letter).—The Synopsis of this paper leads one to believe that the author will perform a needed service in presenting sound reasons for revising the Joint Committee Report on matters relating to columns. After reading practically all Mr. Tucker's paper, the writer is not convinced that designing would be on any better basis if his recommendations were adopted than it is on the basis of the Joint Committee Report.

The Joint Committee Report of 1909 allowed 450 lb. per sq. in. on concrete in a rodded column, but excluded a protecting shell, 1½ in. thick, the concrete in a 10-in. column (2 000-lb. concrete) being safe for 22 050 lb.

The report of 1912 allowed 450 lb. per sq. in., and excluded a 2-in. shell, the same column being safe for 16 200 lb. on the concrete.

The 1917 Report allowed 450 lb. per sq. in., but did not exclude the shell, the same column being safe for 45 000 lb., or twice that allowed under the 1909 Report and nearly three times that allowed under the 1912 Report. With exactly the same data on which to base judgment, totally different standards were evolved.

The first two reports of the Joint Committee permitted the use of plain concrete columns up to twelve diameters. These same reports indicated clearly that the function of ties in rodded columns was merely to hold the upright rods in place while the concrete was hardening. Any kind of thin wires would answer this purpose. The later reports specified rods of a certain size for ties, and a spacing not to exceed sixteen diameters of the longitudinal steel.

These facts, together with a clearly defined dissent from the standards of the Joint Committee, referred to by the author on page 1089, suggest the need of an engineering commission to investigate the subject of columns. This commission should have open and public proceedings, should listen to any one wishing to be heard, and should render specific answers to arguments against the standard.

It is difficult to understand the method by which the author undertakes to prove his case against existing standards and sustain his substitution therefor. There are parts of the paper that seem to contradict other parts: The author states, on page 1089: "Even those engineers who most favor this type of column [the rodded column], cannot agree on the value of the rods nor can they prove that they have a definite value." Then, on page 1116, he states: Longitudinal reinforcing rods add definite strength to the column."

On page 1140, in the criticisms of the "1921 Tentative Specifications", the author states:

"The working stresses, in all cases, are lower than the safe calculated stresses, provided the concrete in the column is equal to the nominal concrete strength on which design has been based, * * *."

* Structural Engr. (Robert W. Hunt & Co.), Pittsburgh, Pa.

In the next paragraph he states:

"For the strengths to be expected from a nominal strength concrete, the permissible stresses of the 1921 Tentative Specifications are much above the safe calculated stresses for many possible selections of variables."

Surely working stresses and permissible stresses are the same; but how can such stresses be lower than the author's "safe calculated stresses" in all cases and much above the safe calculated stresses in many possible cases.

On page 1099, he states that it is impossible to determine the true stress in the rods; and later, in the same paragraph: "It will be shown subsequently that the rods develop a very constant strength."

It is stated on page 1129 that: "A unit volume of concrete has the same ultimate strength no matter where it is placed." On page 1096, a formula is given for the "increase in strength of concrete due to spiral reinforcement", thus proving that if the volume of concrete is placed inside a spiral it will be of much greater strength. The author states on page 1098 that "the spiral increases the strength of the concrete in a definite manner." Surrounded by restraining concrete, and carrying a bearing load; 1 cu. in. of concrete has many times the compressive strength of a free cube; so that 1 cu. in. of concrete, wherever placed, has by no means a certain and fixed value as a resistance to compressive stress.

On page 1136, the author says:

"The contention is untrue that, in the process of destruction of the [rodded] column, the rods buckle and cause the surrounding outer layer of concrete to scale, thus starting and aiding the failure of the column. The phenomenon is a phase in the failure of the testing of this type of column, and represents the buckling of the rods after the maximum strength of the column and concrete has been passed and the concrete has broken, allowing the rods to buckle outward."

Apparently, his conclusion here is based on the suddenness with which the whole column breaks down. On page 1118, he refers to the suddenness with which the Quebec Bridge collapsed. Would he say that the entire Quebec Bridge reached its ultimate strength in every part and that no one part failed first, thus causing the whole collapse?

In addition to the foregoing contradictions, there are statements and attitudes toward engineering problems that are difficult to harmonize with fact and logic.

On page 1127, the author states: "Increase in the factor of safety, a result of including too many and extraneous factors, does not give increased safety," and, again, in the same paragraph, "The smaller the factor of safety the safer a structure becomes." Is it the actual dimensions and type of design that give stability to a structure, or the correctness of the logic of the designer—the idea in his mind?

The author's statement about reliability is difficult to connect with any well-ordered set of facts. When reliability is mentioned, one thinks of something dependable, not likely to fail suddenly and without warning; but the author rates high in the scale of reliability the very things that are the most treacherous. He states, on page 1118:

"* * * the suddenness and violence of the failure or the length of time required for failure to occur in the testing machine is no indication of the strength reliability of the material or structural element."

On page 1142, he states: "The rod and spiral reinforced concrete column is shown to be almost as reliable as the rodded column." This sounds like irony. Hundreds of rodded columns have failed in the general collapse of more than a score of structures; and no structure having spirally reinforced columns has collapsed. The author states, on page 1141, that the spirally reinforced column is less reliable than the plain concrete column. Then why use reinforcement in columns?

On page 1082, the author states, under "Unreinforced Concrete Columns": "Applicable test results on only twelve columns are available", and, among these, he selects seven out of nine made by Messrs. McKibben and Merrill. Professor Talbot reports* tests on nineteen plain concrete columns, and other experimenters have made tests on such columns. Why this meager selection of data, when an attempt is made to establish a principle?

On page 1092, the author states that the rods in a column "have a strength of 39 700 lb. per sq. in. for a column of infinitely small length". A vertical rod, even of finite length, will have an ultimate strength that no testing machine could measure. Throughout the paper, the author ignores this vital fact, that thin disks of any material are enormously stronger than cubes or blocks of ordinary dimensions. It is incorrect, therefore, to mention the strength of any column of infinitely short length.

The whole tenor of the paper seems to be an effort to predict the laboratory result when columns are made or tested in the laboratory or under the most perfect conditions. Is the laboratory used merely for the purpose of discovering and predicting what results may be expected in any given series of laboratory experiments subsequently made; or, is it a tool of the designing engineer; and are results to be interpreted with a view of helping him to solve the problems of stability? In the laboratory, columns are tested with a balanced or symmetrical load. In practice, a column with a balanced or symmetrical load is exceedingly rare, as rare as a fixed-ended column; and this, again, is practically the only kind of column tested in the laboratory.

In the author's mind, reliability seems to be the probability that a given column will develop a certain predicted strength in a laboratory test, and no more. (He rejects some spiral column tests because the results are too high.) In the engineer's mind, reliability is something to be depended on, even if materials and manipulation in the field are not perfect. The author finds most reliable the things that many engineers have found to be least reliable. The rodded column which he classes high in the scale of reliability, has been the subject of more failures than any other structural element.

Toughness, the quality of withstanding overload without distress, of taking unequal stress, either because of unbalanced loading or of inequality of the concrete in different parts of the column—this quality has been the saving of

* *Bulletin No. 20*, Univ. of Illinois, Eng. Experiment Station.

many a structure none too well designed. Its opposite, brittleness, has caused the failure of a large number of structures. The author, however, in his scale of reliability, rates the most brittle columns higher than the toughest.

It is futile to say that one must eliminate irregularities of loading or irregularities in the making of concrete columns. One might as well try to control the weather as to introduce laboratory methods on the job, and, in fact, the weather has a great effect on the quality of the concrete.

The author ridicules the common use of a factor of safety, and attempts to show that it is not based on scientific principles. The phrases, "scientific basis of engineering knowledge", "rigid control of the making and placing of concrete", "advance in technique", sound well; but if there is no margin to cover uncertainties in execution and unknowables in design, what is to be done, if a batch of bad concrete is placed in a rodded column, or some concrete is affected by frost? The engineer who does not exert himself to make his structures "fool-proof", not only by using a factor of safety to cover contingencies, but also by using a design that will produce tough columns, is likely to experience serious failures.

Important as is the factor of safety, to insure a substantial structure and to make every part capable of taking its load safely, it is yet more important to care for the general stability of a structure by adopting a proper design, members with proper reliability, and a structure in which, on the whole, no principle of equilibrium is violated. Factors of safety did not enter into either of the failures of the Quebec Bridge.

Reinforced concrete failures, contrary to the statement on page 1130, have not been due to inferior concrete, except where such concrete may have started the failure, and brought on the collapse of a brittle and unreliable structure. In these structures an increased factor of safety, exhibited by merely adding more brittle concrete to improperly designed columns, would have had little if any benefit.

If laboratory tests, instead of being made on isolated members, ideally constructed, and conditioned, were made on two or on four columns, monolithic with girders, the inherent brittleness and unreliability of the rodded column would have been manifested long ago. One test was made in this manner, and the columns failed at about 400 lb. per sq. in. Failures have shown that the stresses in rodded columns were as low as 150 lb. per sq. in.

It will doubtless be argued that the failures of columns at low unit stresses are due to eccentric loads. It is easy for the experimenter and specification writer to say that eccentric loading should be taken into account in the design, but it cannot be done. The writer does not insist on it, except where the eccentricity is obvious, as where the plane of a girder is to one side of the column axis, or where a steel girder rests on a bracket and has no web connection to the column.

Given the simplest case, of a steel girder supported by two columns to which it is riveted by a web connection: A practical designer will design the girder for a span equal to the distance from center to center of the columns,

and will design the columns for the direct load. He will be right, and his structure will carry its designed load; but the theoretical designer will doubtless compute a bending moment in each column, and design the girder for the shortened span. The former method is the way designing is done, and probably always will be done, in spite of all the books and specifications that may be written.

Suppose these same parts are to be designed in reinforced concrete: If the columns are hooped, the same method of computing the bending moment in the girder and the direct load in the columns may be used with safety. If the columns are of the rodded, brittle, unreliable type, the designer who does the same thing does it at his peril, especially if the girders are long or shallow. The latest large failure of record, that at Salina, Kans., is an example.

The author excuses all low values in column tests, and explains all high values on the convenient basis of the great variation in the strength of concrete. By thus allocating any desired value to the strength of the concrete, of course, a reinforcing value of very definite amount can be found for upright rods, no matter what strength the column may show in the test. Among Professor Talbot's tests, in *Bulletin No. 10* previously mentioned, one plain concrete column stood nearly 50% more load than one rodded column; and if four tests had not been made, the average of the plain columns would far exceed that of the rodded so-called reinforced concrete columns. If the author can make out of this an argument that upright rods reinforce a column, he is welcome to any assurance it may give him for his laboratory work; but he should not ask designers to risk valuable lives and property on that kind of assurance.

Of the piers tested by the U. S. Bureau of Standards,* the hooped piers having 1% of longitudinal steel are, on the average, only 1½% stronger than those having no longitudinal steel, instead of 15% according to standard rules. One hooped specimen with no longitudinal steel is stronger than any of those with 1% longitudinal steel, and is exceeded by only one of the group with 2% longitudinal steel. One hooped specimen with 6% longitudinal steel is only 10% stronger than a specimen with no longitudinal steel, instead of 90 per cent. Of the rodded piers, the group with 2% longitudinal steel is weaker than that with 1 per cent. There is no doubt that if these specimens had been columns instead of piers, the results would have been more erratic. If these facts demonstrate anything, it is, to quote the author, that engineers who favor the rodded column cannot prove that the rods have any definite value. Attempts to harmonize such erratic results by arbitrary shifting of concrete values should be anything but convincing, especially to men who are responsible for the safety of structures.

In dealing with spiral reinforced concrete columns, the author fails to recognize the one thing that explains the added compressive strength which he finds to exist in the concrete. He even omits the data by which this added strength might be judged or duplicated. The writer refers to the pitch of the coils. It is not sufficient to say that a column is reinforced with a coil, nor

* Published in *Proceedings*, Am. Concrete Inst., February, 1915.

to give the area of the coil. The pitch of the coil is of great importance. In a hooped column, the succeeding hoops make of the column a system of superimposed flat disks. It is well known that a flat disk of any material, such as concrete, is of very much greater strength than a cube or a cylinder two diameters in height. A cube of lime mortar will stand very little compression, but a thin joint of the same mortar will stand a heavy pressure, and the thinner the joint the greater its carrying capacity. This effect is independent of the lateral confinement, except that which is afforded by the friction of the end planes. In the hooped column, the hoops offer a corresponding confinement. In the spiral reinforced column, a somewhat similar action takes place, and the pitch of the spiral is of great importance. In the tests shown on page 1094, nearly all the test columns have about 1 in. pitch in the spiral. The deductions made by the author, therefore, are of limited application. Columns with a different pitch in the spiral might show very different strength.

When groups of tests have to be excluded to make a formula agree, it does not speak well for the formula. If these groups were included, it would be seen that the percentage of spiral reinforcement has little to do with the added strength of the concrete. Some columns with one-half of 1% spiral reinforcement have a greater increase in strength than others in which the reinforcement is only 1 per cent.

Spiral reinforcement corresponds to lattice in steel columns. Steel columns would be deficient, if the lattice system was inadequate, but the area of the lattice-bars is never included in the area of a steel column, nor is the calculated strength of the column varied with the kind of lattice. The lattice, however, must be sufficient.

Thus, there is doubtless a balanced proportion where the coil area and pitch are in conformity, and it is improper to attribute strength to a column varying with the coil area. If experimenters would endeavor to discover the proper balance between the coil area and the pitch to produce a well-proportioned column, it would be of great benefit to the Profession.

The old, discarded formula of Considère and the author's substitute are both wrong, because they would vary the strength of the column with the varying area of the hooping, when there is nothing in the compression area between the hoops but concrete (and the upright steel, when it is used); and the nature of these is not altered by adding to the steel area. Like the lattice in a steel column, the coil area should be sufficient.

The proper way to proportion a column is to use a unit stress well within what a plain concrete column would stand under a central load, as this, by many tests, is proved to be the point where the column will begin to fail, no matter how it is reinforced. It is the outer shell that governs in the matter of unit stress.

The author belittles the contention that shrinkage of the concrete in a rodded column causes high initial compression in the rods, which buckle out and spall the concrete, thus causing the failure of the column. He states that the rods do not buckle until the whole column has reached its ultimate

strength; but how can he prove this? He cannot deny that in many tests the rods buckle and push out the concrete. Simultaneously with this action, in a heavily loaded column, the center of the column would have to crush. The author will have to exhibit some tests in which the columns broke by splitting and the rods were left embedded in the concrete and straight, if he wishes to satisfy anybody that the rods do not buckle and push the surface concrete out, and thus cause the failure, in these cases. In the Edison Building, many rods buckled out and pushed off the concrete, and the columns were still standing and carrying their load. Some of these rods were in the middle of the side of the column. This happened even in rooms where the heat was not sufficient to melt insulation on electric wires.

It is far from convincing to mask all irregularities and account for every freak result by the blanket statement that the concrete of the column is of irregular and unknowable strength, as the author has done.

The von Emperger type of column, with its cast-iron core of special, high-strength cast iron, and its hooped concrete shell, would better be left as a laboratory curiosity. There is no practical way to make a good splice in the cast-iron core, either for high compression or for bending, to impart toughness to the system. Shop milling is not sufficiently accurate for this purpose. Flanges would break the continuity of the hooped shell. Besides, it is difficult to connect beams and girders to columns of this type.

On page 1141, the author states: "It is immaterial how a structural element fails", and, on page 1131, he states:

"It is immaterial * * * whether the rods of a longitudinally reinforced concrete column cause the failure of the column or whether the failure is independent of the action of the rods."

On the same page, he intimates that the ground for this non-concern is because the same phenomena occur in the test columns. The engineer is vitally concerned with what makes a structural element fail. In no other way can he avoid the recurrence. Suppose, for example, that it was established that the rods in a rodded column did cause the failure by buckling, the simple expedient of using close-spaced hooping would prevent failures of this type.

E. S. Martin,* Assoc. M. Am. Soc. C. E. (by letter).—Although Mr. Tucker's paper presents the most thorough study of reinforced concrete columns that has come to the writer's notice, he will discuss only one point at this time.

It is recommended that unit stresses be reduced 40% if field cylinder tests are not made. This is an important point affecting very materially the size and costs of columns, and yet, apparently, this recommendation is not supported by anything in the paper.

Of what value are field cylinder tests? It is the usual practice to cast two or three pairs of cylinders from a day's run consisting of 200 to 400 batches. In testing open-hearth steel at the mill, the engineer requires tests to be made of each charge or heat, and would not regard very highly a report of tests

* Secy.-Treas., James A. Wickett, Ltd., Toronto, Ont., Canada.

made from one heat out of a hundred, although that is all a field cylinder test of concrete is.

In the writer's judgment, field cylinder tests are indications only of the quality of the concrete that can be made from a particular combination of aggregates and cement; and it is only necessary to make these tests for fresh supplies of either.

GEORGE PAASWELL,* M. AM. SOC. C. E.—The author makes no pretense that this paper is a contribution to column literature *per se*, but it is unquestionably a most stimulating contribution to reinforced concrete, from the standpoints of both design and experiment. As the reinforced column is by definition a compression member, it is the usual practice to design it by the accepted column formula, introducing such coefficients as experiment indicates, and making the formula, of course, a function of the slenderness ratio. As a matter of fact, it is seen by a study of the stress-strain diagrams (and it has been pointed out before) that, with the slenderness ratio of the usual concrete column practice, the column strength is not a function of $\frac{L}{D}$.* This leads the speaker to suggest the abandonment of attempts to mould the design formula to fit the column theory, and that a frank attempt be made to design the member as a heterogeneous block subjected to a principal compression stress, its failure being due to shear, internal friction, or otherwise, along experimentally determinable surfaces. The speaker suggests this method, in spite of the apparent complexity introduced into column design, with the hope that some light may be thrown on the, at present, hopeless muddle of spirally reinforced columns.

It is a matter of no great mathematical hardship to start from the comparatively simple elastic equations of isotropic solids and derive design equations. It is, however, a matter of grave decision as to what modifications must be introduced into the formulas to have them meet the variations of the given material from the ideal properties of an isotropic solid. To take a spirally reinforced column as an example of an isotropic solid makes extremely approximate the resulting equations of elastic action. This is merely by way of gentle criticism, however. One must start with the isotropic conditions and then modify the analysis to meet the material variations.

The author would do well to emphasize the qualitative worth of the analysis rather than the quantitative. The Poisson ratio always breeds confusion and attendant distrust of analysis dependent on its determination. The ratio, presumably, is restricted to values between 0.25 and 0.5. As a matter of fact, there is no direct way of obtaining, experimentally, the value of the ratio. It can only be obtained by indirect methods. This is probably the reason for the great variation in its determination. Thus, Mr. A. E. H. Love† notes that Professor Voight, in obtaining the elastic properties of certain crystals, having axes of isotropic action which are well defined, finds a value of the Poisson ratio of $-\frac{1}{7}$. The famous ratio is certainly a double-edged sword, and designers do well to hang very little analysis on it.

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* Section Engr., Public Service Comm., First Dist., New York City.

† "Treatise on the Mathematical Theory of Elasticity", First Edition, Vol. I, p. 96.

A Japanese investigator, Chido Sunatani,* has pointed out that the mechanics of failure are but little understood, and, as a result of experiments, has indicated a method of design predicated on a combination of shearing and internal friction action. Without vouching for the accuracy of these equations, aside from noting that the phenomena of failure are consistent with the investigator's equations, it seems to the speaker that more success could be obtained by either testing or mathematically analyzing the reinforced column along such lines, than by making a fetich of the column formula.

The matter of reliability is so ably and concisely put, and its application is so obvious, that one wonders that it has not appeared in print before. One cannot emphasize sufficiently the Society's debt to Mr. Tucker for his keen analysis of the application of statistical methods to the study of tests. It is true that an average test strength is meaningless until one knows the variations from this mean strength.

In the matter of working stresses, it must be always borne in mind that the concrete receives very little sympathy from the average contractor. In matters of decision, it is the plant and not the concrete that receives the benefit of the doubt. On the other hand, a cylinder test is not usually a fair and typical indication of the strength and carrying capacity of the concrete structure. The building contains no structural unit stressed like the cylinder. Briefly, the cylinder test is not an absolute measure of the value of the concrete, but merely a relative one. The speaker hopes to read, some day, of extensive tests made on structural frames, in which the units are loaded in accordance with engineering practice, and not in accordance with the highly ideal conditions of "laboratory" columns and beams.

JACOB FELD,† JUN. AM. SOC. C. E.—There are two points in this paper which the speaker believes are not substantiated by experiment: The reduction formulas for column length, and the action and failure of spirally reinforced columns. The concrete columns used in practice seldom have an $\frac{L}{D}$ ratio greater than 15. The following tests (including those quoted by the author) of plain concrete columns show that the $\frac{L}{D}$ ratio has practically no effect on the allowable unit load, within the usual maximum $\frac{L}{D}$ ratio. It is not fair to compare the strength of a test column with that of a single test cylinder. It is better to compare the strength of columns with the average strength of the concrete used, as determined by the average of all the cylinders tested. The argument in favor of the method used by the author, that cylinder and column come from the same batch of mix, does not carry as much weight as it would in testing steel columns, comparing the column with a test specimen made from the same ingot. Such uniformity as that obtained in steel manufacture cannot be expected in concrete, where five or six men using equipment

* The Technology Reports of the Tohoku Imperial Univ., Sendai, Japan.
† Brooklyn, N. Y.

costing about $100 may constitute the entire construction as well as production unit. Comparison with the product of the steel mill is impossible.

In considering the M columns in Table 1, the low values for the columns with a slenderness ratio of 16.6 are probably due to the exceptionally high test cylinder for that case. A comparison with the average cylinder values, including the tests at the University of Texas,* made by F. E. Giesecke, M. Am. Soc. C. E., in 1919-20, at the University of Illinois,† at the University of Wisconsin,‡ and the Aberthaw tests at the Watertown Arsenal (1897), is shown in Table 14.

TABLE 14.—EFFECT OF SLENDERNESS RATIO. PLAIN CONCRETE COLUMNS.

Source.	Diameter, in inches.	$\frac{L}{D}$	Ultimate Stress, in Pounds per Square Inch.			Ratio:
			Cylinder.	Cube.	Column.	
M.....	14	4.2	2 032	.....	2 070	1.02 to cylinder.
	14	8.5	2 032	.....	1 767	0.87 " "
	14	16.6	2 032	.....	1 747	0.86 " "
W_{466}.....	10.5	9.72	2 417	.....	2 600	1.08 " "
A. C. I., 1915	20.9	7.0	2 647	.....	2 745	1.10 " "
Illin. *10, 20*	12	10.0	.....	2 200	.....	(2 mos.) 0.78 to cube, 2 cols.
						(6 mos.) 0.90 to " 6 "
W_{300}.....	12 by 12	10.0	.....	2 300	.....	0.88 to " 4 "
Texas	3 by 6	5 to 25	(5 sets of 30 columns each, wet mix.)			Little variation.
Aberthaw..	12 by 12	2 to 14	(hand-mixed concrete.)			1.00 ± 0.076.

Before drawing any conclusions from Table 14, the following notes, from the reports of these tests, should be considered:

"It is noticeable that there is but very slight, if any, decrease in strength with the length of the specimen, and this difference may probably be due to the fact that the longer columns were tested at earlier periods."

"The longest columns were tested in 35 days, the shortest in 47 days."§

"Strength of these concrete columns varied only slightly with the slenderness ratio, which, however, did not exceed 25; but it also shows that a variation in the consistency of the mix or a variation in the method of the placing of concrete may produce a very great variation in strength, more than 100 per cent. * * * The strength of the column is almost independent of the slenderness ratio."‖

"In considering these tests it must be borne in mind that the specimens [columns] were very carefully made."¶

Remembering, also, that the unit strength of cubes is greater than that of cylinders, there is no doubt that every one of these tests shows that the unit strength of columns is practically independent of the $\frac{L}{D}$ ratio. The greatest

* *Engineering News-Record*, v. 89 (1922), pp. 274–276.
† *Bulletin Nos. 10* and *20*.
‡ *Bulletin No. 300*.
§ Aberthaw tests at the Watertown Arsenal, 1897.
‖ *Engineering News-Record*, v. 89 (1922), pp. 274–276.
¶ Univ. of Wisconsin, *Bulletin No. 466*, p. 14.

reduction is 0.14 for the M column with $\frac{L}{D} = 16.6$, the other tests show that $\frac{L}{D} = 8.5$ gave an unusually low value. A comparison of the author's reduction formulas with those of various specifications may be of interest, and is shown in Table 15.

A.—French regulations (1906): $P' = \dfrac{P}{1 + \dfrac{k}{10\,000}\,\dfrac{L^2}{r^2}}$.

*A*1.—For round-ended columns, k is 1.0; no reduction for $\frac{L}{D} = 20$.

*A*2.—For fixed columns, k is 0.25; no reduction for $\frac{L}{D} = 20$.

B.—German regulations (1905-07): for $\frac{L}{D}$ greater than 18, and a factor of safety, $s = 10$, $P' = \dfrac{\pi^2\, E\, I}{s\, L^2}$.

C.—English (1909): $\frac{L'}{D}$: 15 18 21 24 27 30

$\frac{P'}{P}$: 1.0 0.8 0.6 0.4 0.2 0.0

C 1.—For round-ended columns, $L' = L$, the length of the column.

C 2.—For fixed columns, $L' = 4\,L$.

D.—Swiss (1909): for $\frac{L}{D}$ greater than 20, $P' = \dfrac{45}{35} \cdot \dfrac{P}{1 + \left(\dfrac{L}{100\,r}\right)^2}$.

E.—Austrian (1911): for $\frac{L}{D}$ greater than 30, $P' = P\left(1.72 - 0.024\,\dfrac{L}{D}\right)$.

F.—Los Angeles (1915): for $\frac{L}{D}$ greater than 15, $P' = P\left(1.6 - 0.4\,\dfrac{L}{D}\right)$.

G.—Joint Committee, Tentative Specifications (1921): for $\frac{L}{D}$ greater than 20, $P' = P\left(1.33 - 0.033\,\dfrac{L}{D}\right)$.

H 1.—Tucker's formula for plain concrete columns: $P' = P\left(1 - 0.171\,\dfrac{L}{D}\right)$.

H 2.—Tucker's formula for columns with longitudinal steel only:

$P' = P\left(1 - 0.0265\,\dfrac{L}{D}\right)$.

H 3.—Tucker's formula for columns with rods and spirals:

$P' = P\left(1 - 0.0183\,\dfrac{L}{D}\right)$.

Therefore, it seems that the author's recommendations for reductions in the unit loads, in order to allow for the $\frac{L}{D}$ ratio, are unduly harsh. The speaker

was also impressed by the lack of consistency in Table 9. Certainly there is no reason for allowing a less reduction for plain concrete columns than for the reinforced columns, as such reduction is meant to allow for the tension at the outermost fiber caused by flexure.

TABLE 15.—RATIO OF COLUMN STRENGTH TO CYLINDER.

$\frac{L}{D}$	*A* 1.	*A* 2.	*B*.	*C* 1.	*C* 2.	*D*.	*E*.	*F*.	*G*.	*H* 1.	*H* 2.	*H* 3.
10	.....	.....	....	....	....	....	....	....	....	0.829	0.735	0.817
11	.....	.....	....	....	....	....	....	....	....	0.812	0.708	0.809
12	.....	.....	....	....	....	....	....	....	....	0.795	0.692	0.780
13	.....	.....	....	....	....	....	....	....	....	0.778	0.655	0.762
14	.....	.....	....	....	....	....	....	....	....	0.761	0.629	0.744
15	.....	.....	....	1.0	....	....	....	1.0	....	0.743	0.602	0.725
16	.....	.....	....	0.93	....	..	...	0.96	....	0.726	0.576	0.707
17	.....	.....	....	0.87	....	....	....	0.92	....	0.709	0.549	0.689
18	.....	.....	1.0	0.80	....	....	....	0.88	....	0.692	0.523	0.671
19	.....	.....	1.90	0.73	....	....	....	0.84	....	0.675	0.496	0.652
20	.....	.....	0.81	0.67	....	....	....	0.80	....	0.658	0.470	0.634
21	0.586	0.850	0.74	0.60	....	....	....	0.76	0.63	0.641	0.443	0.616
22	0.563	0.838	0.67	0.53	....	....	....	0.72	0.60	0.624	0.417	0.598
23	0.542	0.825	0.60	0.41	....	....	....	0.68	0.56	0.607	0.390	0.579
24	0.521	0.812	0.57	0.30	....	....	....	0.64	0.53	0.590	0.364	0.561
25	0.500	0.801	0.52	....	....	1.0	....	0.60	0.50	0.572	0.337	0.542
30	0.410	0.735	0.36	....	....	0.95	1.0	0.40	0.33	0.487	0.205	0.451
40	0.273	0.610	0.20	....	....	0.79	0.86	....	....	0.316	.....	0.268
50	0.200	0.500	0.13	....	1.0	0.64	0.52	....	....	0.145	.....	0.095

Two phenomena which concern directly the failure of spirally reinforced columns have not been considered by the author: The method of failure of concrete, and the contraction in volume on drying out. Concrete specimens, whether they fail in compression, tension, or shear, may fail either by the separation of the individual units making up the concrete (grains of sand or stone), due to the overpowering of the cohesive forces in the cement, or by rupture of the individual grains. From observation of several thousand specimens tested under the speaker's supervision in the concrete laboratories of the University of Cincinnati, he is convinced that failures of the latter type may be disregarded, especially in considering the strength of the usual concrete. Failure of concrete columns is either by flexure or by shear; in either case, the column fails because of the low tensile strength of the concrete. For the tensile strain ruptures the extreme fiber in flexure, and simple shear is a tension along one of two rectangular axes, with a simultaneous compression of equal magnitude along the other. Failure, therefore, occurs whenever the tensile stress overcomes the adhesion produced by the cement. After such failure occurs, the body is no longer a homogeneous solid, even though the separation of the parts may be by an infinitesimal distance. Such failure is reported by several of the experimenters:

"Generally the first sign of failure in the columns appeared in the form of longitudinal cracks, usually occurring from 0 to 2 ft. from one end, although sometimes extending the entire length." (Aberthaw.)

"No cracks or other signs of failure were visible until the maximum load was reached, except that in Column 7*a* a small longitudinal crack was seen in the shaft just below the head at a load of 2 190 lb. per sq. in. At the maximum

load longitudinal cracks developed all around the shaft of the column below the top of the head and extended down over the upper quarter of the length of the shaft. The load at once fell off 20% or more."*

"In some cases no cracks were visible below the maximum load. In general, the load fell off considerably a half minute or so after the maximum load was applied."†

It has been settled definitely that concrete contracts as it sets in air. A. T. Goldbeck, Assoc. M. Am. Soc. C. E., establishes the contraction at a maximum of 0.0005 at 3 months.‡ The importance of this contraction in the spirally reinforced column is the setting up of initial stresses in the concrete and a compression in the spiral. Considering the effect on the concrete, and assuming that the deformation is equal in all directions, the dilatation, σ, equals $s_x + s_y + s_z = 3s$, for an isotropic body, neglecting infinitesimals of higher order than the first. Therefore, s is $\frac{0.0005}{3}$, or 0.00017. This value is probably excessive, for it produces a rather large stress. The normal unit stress, p, causing this dilatation is $(3\lambda + 2\mu)\frac{\sigma}{3}$, where μ is the modulus of shear or of rigidity; σ is the dilatation; $\lambda = \frac{2\mu}{m-2}$, $\frac{1}{m}$ being the Poisson ratio. (Poisson and others assumed that $\lambda = \mu$, and obtained $\frac{1}{m} = 0.25$; but this is only a special value.) The relation between these constants and E, Young's modulus of elasticity, is given by the equation: $\mu = \frac{m\,E}{2\,(1+m)}$.

The values of the normal stress, p, for values of E and m, are as given in Table 16.

TABLE 16.

E.	m.	μ.	λ.	σ.	p. in pounds per square inch.
2 000 000	0.25	800 000	800 000	0.0005	667
2 000 000	0.20	833 000	555 000	0.0005	556
1 600 000	0.20	667 000	444 000	0.0005	445

The effect of such an internal stress cannot be disregarded. The important factor is that the concrete shrinks away from the spiral; in addition, the fireproofing outside the spiral puts an initial compression in the steel, the amount of which can be found by the method of computing stresses in pipes under radial pressure, assuming the spiral to act as a steel cylindrical unit. Such action would require a spirally reinforced column to behave in a different way under loadings which cause a strain less than the ultimate, than under higher

* Am. Concrete Inst., 1915, on plain concrete columns.
† Am. Concrete Inst., 1915, on columns with rods only.
‡ Am. Soc. for Testing Materials, Vol. XI, "Contraction of Concrete".

loads. This is easily seen from the tests of such columns. The ultimate strain of plain concrete is less than 0.0015; Walker finds 0.00142 as a maximum strain, with 5 000 as the corresponding stress.* To save space, the tests by Withey† are averaged to give Table 17.

TABLE 17.

Number of columns.	Percentage of spiral.	Percentage of rods.	Type of concrete.	Stress at Strain of:				
				0.0005	0.0010	0.0015	0.0020	0.0025
8	0.5	0 to 6.1	1 : 2 : 3½	1 480	2 510	3 200	3 410	3 540
8	1.0	0 to 6.1	1 : 2 : 3½	1 410	2 430	3 000	3 200	3 300
2	1.0	8.0	1 : 2 : 4	2 600	4 700	6 050	6 380	6 500
6	2.0	8.0, 10.0	1 : 2 : 4	2 400	4 400	5 600	5 670	5 750
4	1.0	0 , 6.0	1 : 1 : 2	2 300	4 250	5 500	6 200	.....
2	1.0	, 5.7	1 : 1¾ : 3¼	1 650	4 100	5 600	6 100	6 450
2	1.0	5.8	1 : 3 : 6	1 750	2 850	3 350	3 500	3 650

It is very noticeable that practically 90% of the ultimate strength of the column is developed by a strain of 0.0015. The speaker believes that the action and failure of spirally reinforced columns is somewhat as follows: As the column sets, the concrete core shrinks away from the spiral, and the outside concrete shell compresses the spiral. As the load is placed on the column, the compressive strain causes a lateral expansion (Poisson's ratio) until a strain of about 0.0010 has occurred. Beyond that strain, the concrete core, as it expands laterally, presses against the spiral balancing and, as it increases, overbalances the initial compression in the spiral. At an axial strain of 0.0015, the concrete has developed its maximum stress and fails; that is, the adhesion is broken. This adhesion is aided somewhat by the tension which now exists in the spiral. There is now a drop in the load as the concrete shifts its burden on the spiral. Failure of adhesion does not occur, at first, along more than one shear plane. Additional loading tends to spread the column laterally, which is prevented by the tight steel spiral, and axially, which is opposed by the friction between the separated surfaces. This action is very similar to that of a loaded cylinder of sand encased in a sheet of paper. Axial loads tend to spread the sand particles, but a surprisingly large load is required to cause failure. The speaker at one time had occasion to use paper cylinder forms for concrete test specimens, and noted an increase in strength in the specimens tested with the forms over that of the cylinders made in steel forms.

To quote from the 1915 Report of the American Concrete Institute, the final step is:

"As the maximum load was approached, the spiral stress was no longer uniform around the circumference and it varied at different parts of the length of the column. Finally at some point the maximum strength of the spiral reinforcement was reached or the action of the column became very far from uniform, and the maximum column load which could be carried was attained."

* *Bulletin 5*, Structural Materials Laboratory, 1920.

† Univ. of Wisconsin, *Bulletin 466*.

From the foregoing considerations, the theory of elasticity* cannot be used unless there are taken into account: The end conditions, the initial stress in the concrete before loading, the expansion of the concrete bringing the spiral into play, and the incipient separation of the concrete at a strain of 0.0015. Beyond that point, the mass is so far from homogeneous that the theory of elasticity cannot apply.

The author has refused to admit the accuracy of the measurements which gave an evident Poisson's ratio greater than 0.5. In an isotropic body that would be impossible. He claims that the method of measurement, the axial strain being measured over the entire length and the lateral strain localized, is the reason for such values. In the American Concrete Institute columns, lateral and axial strain at twenty points on the concrete and on the spiral were determined. The following gives the average of all the readings in the test of Column 3*b*, 20.75 in. in diameter, 1 : 1½ : 3 concrete, 1.89% vertical and 0.94% spiral steel, and no fire-proofing cover:

Axial unit load.....	3 500	4 350	5 000	5 450	5 800
Longitudinal strain.....	0.001	0.0014	0.0025	0.0035	0.0050
Lateral strain.....	0.0002	0.0003	0.00075	0.0015	0.0027
Poisson's ratio.....	0.20	0.24	0.30	0.43	0.54

Poisson's ratio increases very rapidly after the axial strain has passed 0.0010; in other words, we are no longer dealing with a solid. Too much reliance cannot be placed on the theoretical investigation, even if the stress is below the ultimate, because concrete is much more heterogeneous than is usually assumed. This can be seen best from the difficulty of determining the constants of the material.

It has been proved that E is a variable depending on the conditions under which the concrete was made. Withey reports that at one-fourth the ultimate stress, n. or $\left(\frac{E_c}{E_s}\right)$, the E_s being fairly constant varied from 9.2 to 15 in the columns tested. At the yield point, the variation was from 14 to 30. The $\frac{1}{m}$ was 0.090 to 0.085 at one-fourth of the ultimate stress, and varied with the load. The shear modulus is a function of E and m; the bulk modulus is a function of the shear modulus and m. The dilatation is the ratio of the normal stress to the bulk modulus.

Before the proper theory for columns can be developed, a sufficient number of these "constants" of concrete must be determined experimentally, in order to be able to evaluate every one of them.

The author is to be complimented on his exhaustive treatment of the subject, and the extremely interesting form and manner in which it is presented. It opens up several new views and methods of attack which will lead to valuable results in placing concrete design on a more scientific basis.

A. W. Buel,† M. Am. Soc. C. E.—A paragraph on page 1082 reads, in part: "* * * this type of column is not used in practice. * * * Applicable test

* See pp. 1104-1106.

† Cons. Engr., New York City.

results on only twelve columns are available." The author's explanation of this statement is that: "They are forbidden by the code." As his opening statement reads: "The purpose of this paper is primarily to determine, by scientific methods, values of safe working stresses" and refers to the Joint Committee's specifications, but not to any "code," his explanation seems to be inconsistent, and, in the speaker's opinion, the limitation is unscientific, restricts the application of the results, and reduces the value of the paper.

A prominent engineer has said that we have to have codes, on account of the ignorant and incompetent designers extant. There is something in that, but do not other professions also need a code as much or more? In the medical profession a really efficient code would have fully as great opportunities of saving lives as in engineering.

Codes are subject to revision and radical changes, as are also specifications and "reports of Committees," both of which are the products of compromise and majority vote. If the speaker may paraphrase the description by the late James J. Hill, F. Am. Soc. C. E., of E. H. Harriman's standard box car:

"The Committee's specifications were written by engineers in conference. Each man had his pet mixture, reinforcing system, column formula, etc. So your specification is a composite of engineers' hobbies, not a truly standardized or scientific specification to meet conditions. A scientifically drawn specification may be written which will be far broader in its application and give greater economy without sacrificing safety; but you can bet such a specification will not be written in conference."

Therefore, provisions of codes and specifications should not be permitted to have a controlling effect on research work.

The author could only find twelve tests of plain concrete columns. This is only one among many instances that illustrate how valuable work of only fifteen or twenty years ago is either entirely overlooked or else ignored by present-day workers. Possibly, they think the old experimental work was not sufficiently scientific and accurate to be entitled to serious consideration. Perhaps it is an effect of the World War; but, whatever the cause, research workers would do well to start each new line of work with an exhaustive search for all past experimental results bearing on the subject in hand. In this case, it happens that the U. S. Government publications, "Tests of Metals and Other Materials" made on the testing machine at Watertown Arsenal, for the six years from 1904 to 1909, inclusive, contain reports of tests of 206 concrete columns, plain and reinforced, of which 64 were of plain concrete or mortar. These series are very interesting and instructive. The earlier ones were included in a work by the speaker, published in 1906, which has had wide distribution. They show conclusively that cement is cheaper than steel for reinforcing concrete columns; that a very rich concrete column, without reinforcement, will do the work for less cost than a column of lean concrete reinforced with steel. They also show the low efficiency of hooping for rich concrete. There are also reported in the same volumes nineteen tests of columns with various mixtures, made for the purpose of determining the load sustained by the concrete, and the load sustained by the steel, in columns with longitudinal rods. The author seems to give the impression that these determinations

would be difficult to manage. It would seem to be a comparatively simple matter to make such determinations with extensometers.

In many places, the use of plain concrete columns is perfectly good, and gives the greatest economy, but certainly steel reinforcement should always be used where there is eccentricity of loading, transverse loading, or bending, or where the beam and girder connections and details make the latter continuous with the column, consequently throwing bending into the column, and when wind is carried by bending in the columns. Thus, in city work, where buildings of several or many stories constitute the bulk of the structures, longitudinal reinforcement will be generally necessary, and it may be quite proper to require it by the code, but in many places far afield the requirements are very different.

The restriction of plain concrete columns to a length of six diameters does not seem to be justified. Plain concrete is, or can easily be made, as good as stone, and lengths of nine to ten diameters occur in several of the classical orders. The speaker thinks ten to twelve diameters should be permitted in certain types of structures and under proper conditions.

Referring to Fig. 2, the author gives a "representative line" for the relation of strength of columns with slenderness ratios $\left(\frac{L}{D}\right)$ from 0 to 18, to "cylinder strength." The line does not appear to fit the points plotted from the test records, and is not convincing. Although it would be preferable to have a great many more points to locate such a line with any assurance, as far as they go they certainly indicate a line coincident with the 100% abscissa from $\frac{L}{D} = 0$ to $\frac{L}{D} = 10$, or greater. Moreover, when a larger number of experimental determinations are available, it is not improbable that additional light may be shed on the "discrepancy", mentioned by the author, that his "representative line" does not cross the 100% line at $\frac{L}{D} = 2$, if, in fact, the discrepancy does not entirely disappear. If 100% were used for values of $\frac{L}{D}$ up to 10 or 12, it would be in accord with past practice with other materials.

H. V. Spurr,* M. Am. Soc. C. E.—The author has performed a valuable service in reviewing the various tests on reinforced concrete columns and in presenting in tabulated form an original interpretation of their meaning.

The problem of the designing engineer is to design safely and economically. The formulas for concrete columns as proposed by various committees, and as stipulated in the building codes of various cities, differ greatly in form and in the allowable safe load on columns of the same make-up. Arbitrary factors have been adopted for the value of longitudinal as well as for that of spiral reinforcement, and different values have been given for the concrete itself.

* Chf. Engr., Purdy & Henderson Co., New York City.

Designing engineers would welcome a formula for reinforced concrete columns which would meet with general approval and which would result in both economy and safety.

Economy in column design is largely a question of the design of columns carrying heavy loads in the lower stories of buildings. In such columns large quantities of reinforcement are used, as it is desired to reduce the size to the minimum. In order to design such columns economically, a reasonable stress must be allowed on the concrete, as the percentages of reinforcement are limited for practical reasons. In large columns supporting heavy loads, the cheapest reinforcement is cement; that is, a concrete of high strength and reliability. Strength and reliability are dependent not only on the stone, sand, and cement, but on the control of the proportions and the water content. Columns can never be designed economically until reasonable care is taken to insure a concrete of fair reliability and strength.

Objection is made to the various formulas now in use, in that they do not sufficiently emphasize the importance of securing good concrete, but attempt rather to discount the use of poor concrete. By controlling the water content, and with good aggregate, it is possible to increase the reliability of the concrete and to use higher stresses.

It is reasonable to suppose that spiral reinforcement adds strength to a concrete of a given mix, but this increase should be discounted somewhat in the formula adopted, as cages are not perfectly fabricated and assembled in the field. It would be reasonable to assume that longitudinal reinforcement of large rods well tied by bands or hooping, and encased in concrete, should be capable of taking a safe load approximating that which the same area of steel would sustain in the outstanding legs of a structural column. A reasonable stress for longitudinal reinforcement would seem to be 12 000 lb. per sq. in. For practical or economical limits, 5% of the longitudinal reinforcement should be the maximum.

In building construction in general, repetition of stress can be neglected, due to the large proportion of dead load in the columns. This is especially true in large columns in the lower stories. As the design of columns is based on the strength of concrete at 28 days, it would seem reasonable to assume that the normal increase in strength with age would counteract the effect of time loading within the range of the safe working stresses that would naturally be used without considering this factor.

It would appear, therefore, that the principal elements to be considered in the design of columns, in the order of their importance, are:

1.—The reliability and strength of the concrete.

2.—The age at which it would receive its maximum load.

3.—The increase in strength from a moderate amount of longitudinal reinforcement at 12 000 lb. per sq. in.

4.—The increase in strength due to spiral hooping of mild steel between the limits of three-fourths of 1% and 1½%. This increase in strength should be expressed as a percentage of increase in a safe stress on the concrete in terms of percentage of spiral.

5.—That the ratio of length to diameter would have little effect up to 10, and that above that figure some reduction in the safe stress should be made.

The most important element in the strength of a concrete column, the strength of the concrete itself, can only be determined within certain limits. Tests of a sufficient number of specimens for a given mix, in which the water content had been controlled, have not been made to determine the relation between the strength of the concrete in the columns at 6 months to that of the standard specimens at 28 days. It is reasonable to suppose that the average strength of the concrete in the columns at 6 months would be considerably greater than that at 28 days, and that the range of strength of the columns will not be as wide as that of the specimens. Therefore, if preliminary tests of the concrete mix, proposed to be used for a nominal 3 000-lb. concrete, show that none of the specimens developed less than 2 500 lb., it is reasonable to assume that the concrete in the columns at 6 months will never fall as low as 2 500 lb., provided the materials average well and reasonable care is exercised to control the proportions and the water content. For such a condition, 1 000 lb. per sq. in. on the concrete would not seem to be an excessive safe working stress, which could be increased a certain percentage by the use of spiral reinforcement.

Formulas which are based on a low working stress in the concrete in order to discount poor concrete, but which give a high influence to spiral reinforcement, are unbalanced. Such formulas lead to the use of a maximum quantity of spiral in small columns with a minimum quantity of longitudinal steel, without any serious attempt to obtain good concrete. With large columns, in which the percentage of spiral is limited by the size of hooping available and practicable, a reduction in the percentage of spiral reinforcement necessitates a reduction of stress in the core, which can only be made up by adding a great quantity of longitudinal reinforcement. Thus, economy of design is impossible for the very columns for which it counts the most in cost. Also, in the large columns in the lower stories of a building, the concrete will be several months old before it receives even its full dead load.

Austin H. Reeves,* Assoc. M. Am. Soc. C. E.—For more than twenty years practically no advance has been made in the development of a new and proper type of reinforcement for concrete columns. Most thinking engineers who have had to design concrete structures have recognized the fact that the present methods of column design are faulty. It is easy to design a column to pass the building code of any given city or to meet the requirements of the Joint Committee specifications, yet it is difficult (impossible for the speaker) to justify any of the designs thus made with well known principles of mathematics, mechanics of materials, and common sense. This is said with no lack of respect for any one who has contributed a share—by making tests, writing textbooks, or otherwise—to the present knowledge of the subject.

The title of this paper is erroneous. A reinforced concrete column should be able either to support a greater load before incipient failure or to support

* Newark, N. J.

the same load with greater integrity or at less cost. The speaker will show that the present-day types do none of these things, and, besides, they are unscientific. Therefore, they are not reinforced. (It should be noted that this is true if total volume of concrete is considered—there is no fairness in considering only the core area of one type as against the total area of another, and using the increased ultimate unit stress on this partial area to argue for greater load sustained.)

There are really only three classes of concrete columns: (*a*) plain columns; (*b*) columns containing a mass of steel; and (*c*) reinforced columns. Plain columns need no explanation. The second class includes present-day practice and is a futile attempt to strengthen plain columns. The third class embraces a truly reinforced concrete column on which the speaker has patent rights* pending, and which may be called the mat or "spider" system of lateral reinforcement.

The four headings under which the speaker wishes to develop his remarks are as follows:

1.—A plea for the plain concrete column.

2.—The introduction of a new laterally reinforced system developed by the speaker, which may be called the "spider" system.

3.—Recommendations for different procedure as regards size and test of test specimens.

4.—Proof that the ratio of length to diameter has no place in any concrete column formula.

Mr. Tucker states that: "* * * this type [unreinforced concrete] of column is not used in practice." The statement that such columns are not used in practice is in error, but the speaker believes that even if the plain column was not used in practice, the tests should be of great value if properly made and the results diligently studied, because only in that way can the best type of reinforced concrete column be developed.

The author's next sentence is also incorrect, in that it states: "Applicable test results on only twelve columns are available, and these results, together with data on the columns, are given in Table 1." The results given in Table 1 do not include test data on any of the following thirty-six plain columns which have also been tested, namely, six plain columns, three of which were 12 by 12 in. by 12 ft., one was 12 by 12 in. by 6 ft., one was 9 by 9 in. by 12 ft., and one was 9 by 9 in. by 6 ft., given in *Bulletin No. 10,* University of Illinois; Column No. 1 583 (12.66 by 12.59 in. by 7 ft. 10 in.), which was tested at the Watertown Arsenal in 1904; twenty-one plain columns tabulated in *Bulletin No. 20,* University of Illinois; five plain columns which were remarkably uniform, reported on page 11 of *Bulletin No. 90,* University of Wisconsin; and three plain columns,† tested by H. C. Berry, M. Am. Soc. C. E. The speaker does not mean to infer that this additional number completes the list of available results on tests of plain columns, as he knows of others that have been made.

The speaker will prove that the ratio of length to diameter has no place

* Patent pending, filed February 16, 1923, Serial No. 619 470.

† *Engineering Record,* February 19, 1910, p. 209.

in any concrete column formula. This will show that practically everything on page 1083 of the paper is incorrect, including the curve, Fig. 2, and Equations (3), (4), and (5).

Having discredited practically all the author has said on plain columns, the next step will be to show that, when properly considered, the plain column is the best concrete column produced by any present-day method. Professor Withey has stated:*

"In the table, however, the areas and percentages are based upon the areas of the cores inside the spiral; these values were used in computing the ultimate strengths and stresses at the yield point * * *. This procedure is permissible since the strength of these columns at the yield point or ultimate was approximately independent of this shattered outside shell."

This is one of many authorities who have found that beyond the yield point (which averaged 74% of the ultimate for the thirty-two columns, the test results of which are tabulated on page 42 of *Bulletin No. 466*, University of Wisconsin), the outside shell is useless. It may not be there even when most needed for fire-proofing. It has been found† that four spiralled columns began to disrupt the shell at six-tenths the ultimate load and that the four columns containing both spirals and rods began to disintegrate the outside shell at three-fourths of the ultimate load. On the contrary, plain concrete columns do not show even warning cracks until the ultimate load is reached. It has been stated† that:

"The four 12 by 12-in. plain columns of this series broke at an average stress of approximately 2 070 lb. per sq. in., the maximum variation of a single result from this mean value being only 40 lb. per sq. in., or 2 per cent."

The remarks‡ on the six plain concrete columns were: "Failed without warning cracks", "no cracks appeared before maximum load was reached", and the like. This tends to prove that plain columns do not spall off or even crack until the ultimate load is reached, whereas the shell of the so-called reinforced concrete column spalls off, disrupts, or disintegrates at from six-tenths to seventy-five one-hundredths of the ultimate load—this is what the speaker calls sustaining the load with greater integrity. For out-to-out dimensions of concrete, other things such as mix, density, age, etc., being equal, it has been repeatedly shown that a plain concrete column will hold approximately the same ultimate load as any rodded, spiralled or spiralled and rodded column. Professor Berry§ has recorded the results of tests on nine columns, all 14 in. square and 12 ft. long, three of which were plain, three were rodded, and three were spiralled and rodded. He states:

"The results of the tests impress one with the good showing made by plain concrete and the small increase in strength resulting from the use of reinforcement. The popular notion that the hooped columns are much more desirable because of their greater toughness would seem to be ill-founded because, as can be shown also, by a theoretical analysis, this toughness is not exhibited until after the maximum load is developed, which, if applied by

* *Bulletin No. 466*, Univ. of Wisconsin, p. 38.
† *Bulletin No. 300*, Univ. of Wisconsin, p. 19.
‡ *Bulletin No. 10*, Univ. of Illinois.
§ *Engineering Record*, February 19, 1910, pp. 206–209.

a dead weight instead of a testing machine, would cause almost, if not quite, as sudden a collapse as would occur with either of the other types of columns."

The three plain columns stood an average maximum stress of 2 500 lb. per sq. in. as against 2 580 and 2 810 lb. per sq. in. for the rodded and for the spiralled and rodded types, respectively. Mr. Godfrey has stated* that: "Some plain concrete columns have stood much more ultimate load than some so-called reinforced columns in the same series." He has also stated† that:

"On the other hand, the results of tests on many varieties and styles of plain and reinforced concrete columns, as I have pointed out on other occasions, shows remarkable agreement in one thing; that is, they practically all begin to fail at nearly the same unit load, no matter what the reinforcement or lack of it." And continues: "As illustrating the negligible effect of steel reinforcement on the initial point of failure of columns, I pointed out, in *Transactions*, Am. Soc. C. E., Vol. LXVIII, 1915, page 141, that the average load at 1st failure of set of tests made at the University of Michigan was as follows:

"Plain concrete.....2 250 lb. per sq. in.
Less than 1 per cent. of hooping.....2 250 lb. per sq. in.
More than 1 per cent. of hooping.....2 230 lb. per sq. in."

The equal strength, the greater integrity, and the lower cost of plain concrete columns, as compared with any present-day type of concrete columns, argues for one of two things: Either the use of plain columns or the development of a new and truly reinforced concrete column, such as the speaker has developed. He never has believed that a given size, out-to-out, column could be increased in strength by attempting to increase its compressive strength or by the use of the spiral to suppress lateral deformation. He has believed that, due to the operation of Poisson's ratio, the weakness of concrete in tension, and the great increase in rate of lateral deformation before failure, the development of a scientific system of lateral reinforcement was the proper solution in order to produce a concrete column of maximum strength.

Concrete is strong in compression, fairly strong in shear, weak in tension, and, as regards reinforcement, has its most valuable asset in its bond strength. Steel bars are best in tension and, because of the bond of the concrete, they are able to relieve the tensile stresses of concrete. By making full use of these properties, it seems conservative to predict that a most scientifically designed and spaced system of horizontal "spiders" would develop twice the strength of the plain column.

The "spider" may be of many forms, one type of which is a cluster of six or eight bars radiating from a central ring to very near the outside of the column, with the outer ends either plain, hooked, or other shape. Light wires may tie the rods at intervals to form a mat to be placed in a horizontal position in the column. These "spiders" may be hung from the top of the column form, supported by a proper central shaft, or placed as the column is poured or in any

* *Proceedings*, Am. Concrete Inst., 1921, p. 176.
† *Loc. cit.*, p. 177.

other manner best adapted to an accurate spacing during the pouring of concrete.

On setting, the concrete grips the rods, and as loads are applied the lateral tensile stress is distributed evenly by the bars, the core is restrained in the most rigid manner, and the shell is doing its duty also in sustaining its share of the load until failure. The reinforcement of the column is well protected against heat (in marked contrast to placing the reinforcement within 2 in. of the heat as in the spiral and rodded columns).

The intimation may have seemed excessive that, for a given out-to-out dimension, a properly designed "spider" reinforced concrete column would sustain with equal integrity and with only a little more cost, twice as much as an otherwise equal (regarding age, mix, etc.) plain concrete column. It is well known that cubes give higher test results than cylinders, the ultimate unit stress of the cube is generally about one and one-half times that of the standard cylinder. Few engineers seem to realize that the ordinary test specimens do not give the true maximum ultimate compressive strength of concrete. In order to determine this maximum, the test should be made on very shallow specimens, say, 2 in. high. In specimens of this small height, the restraining effect of the bearing plates of the testing machine is able practically to nullify the effects from the operation of Poisson's ratio. As the average of a sufficient number of tests of cylinders 8 in. in diameter and 16 in. high have been shown to be a good indication of the ultimate strength that a plain concrete column of similar concrete would develop, such tests are alone sufficient as a guide to plain concrete column design.

For the "spider", or for any other truly reinforced concrete column, which can be made to develop the true maximum ultimate compressive stress in the concrete, the ultimate unit compressive stress should be determined by testing specimens not more than 2 in. high. Tensile and modulus of rupture tests should also be made.

The speaker believes it would not be necessary to test prior to each construction job, but that for a concrete of given mix, density, water content, age, class of aggregate, quality of cement, expected character of workmanship, etc., reference might be made to Government tests, Joint Committee tests, or other accepted standard tests which are averages of a sufficient number of specimens to obtain a good forecast of ultimate strength. Such sets of tests would greatly aid the designer and the specification writer.

The speaker will now prove that the ratio of length to diameter has no place in any concrete column formula. Tests indicate that diagonal shear occurs on a plane having an angle of about 35° with the vertical, which shows that when the ratio of length to diameter exceeds 1½, the ratio is no longer important.

A. N. Talbot, Past-President, Am. Soc. C. E., has stated* that:

"It seems hardly necessary to advance the idea here that for concrete columns used in ordinary building construction little attention need be given to the relation between length and lateral dimensions after a length of a few

* *Bulletin No. 10*, Univ. of Illinois, p. 45.

diameters has been reached. * * * Even for 15 diameters we may readily conclude from the calculated results of long column formulas and also from the small lateral deformation found in the columns tested that the difference in strength between a column 15 diameters long and one 5 diameters long is less than the variation among several columns of the same length. * * * For columns eccentrically loaded, the effect of any eccentricity is generally large in comparison with the lateral deflection used in the Euler analysis, and it may be said to be generally independent of the length of the column. Clearly, for conditions of ordinary design, a formula for plain columns or for columns reinforced with longitudinal rods need not include the ratio of length to lateral dimensions."

Professor Talbot and Arthur R. Lord, M. Am. Soc. C. E., have also stated* that, "In other words, the stress taken by the concrete may be considered to be nearly independent of the slenderness ratio of the column, within the lengths tested, * * *."

Messrs. McKibben and Merrill† have stated:

"The average compressive strengths of 8 in. by 16-in. cylinders and 5-ft. columns, 14 in. in diameter, are very nearly equal, while the strength of 10-ft. and 20-ft. columns are approximately equal, amounting to 81% of the strength of cylinders."

Further discussion seems unnecessary as sufficient references and quotations have been given to support the speaker's contention, which might be summarized as follows: That plain concrete columns be considered on their merits; that science be used to the fullest extent possible in concrete column design, which would at once eliminate the present-day types of so-called reinforced columns; that testing of specimens be placed on a more scientific basis; and that the ratio of length to diameter be admitted to have no place in any concrete column formula.

Joel B. Cox,‡ Assoc. M. Am. Soc. C. E. (by letter).—The author's Section III, on "Reliability", wherein the theory of probability is applied to the determination of factors of safety, is of especial interest and importance. It is undoubtedly true that in the strength of a material, as in many other kinds of engineering data, engineers are dealing with a variable of which the mean value is known and for which the probability of its occurrence within any limits may be determined, but for which the exact value in a given case cannot be predicted. The theory of probability and the mathematical methods based thereon furnish the only strictly logical method of dealing with the problem. The writer believes that, in a general recognition of this fact, and in a more widespread use of probability mathematics by engineers, there is opportunity for a great advance in engineering thought. Hence, it seems worth while to add to the author's treatment of the problem.

Such data as those of the strength of a given material can be presented most fully by a graphical or tabular statement of the frequency with which the strength falls within certain limits. If such a diagram as Fig. 14 is presented (or Table 18), showing the distribution of the ultimate strength in

* *Bulletin No. 56*, Univ. of Illinois, p. 43.

† *Proceedings*, Am. Concrete Inst., 1916, p. 209.

‡ Engr., East Maui Irrig. Co., Paia, Maui, Hawaii.

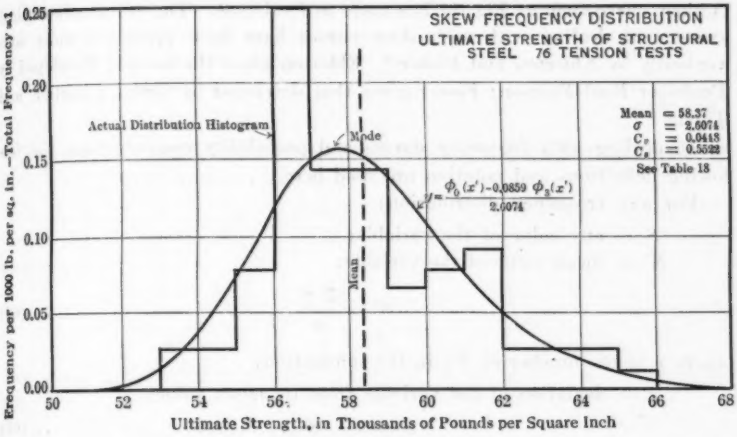


FIG. 14.

tension of 76 specimens of structural steel tested by the Special Committee on Steel Columns and Struts,* it may be said that the data are presented in probability form. Fig. 11 represents such a diagram. It must be borne in mind, however, that the normal or Gaussian form of frequency distribution, represented by Figs. 11 and 15, is by no means the only one possible.

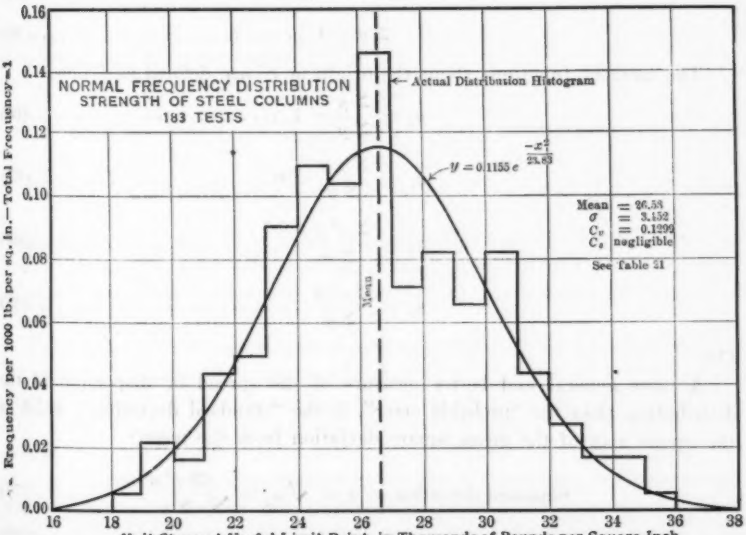


FIG. 15.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1613.

A skew curve such as Fig. 14 is a more general type. The two mathematical systems of dealing with such skew curves have been presented most conveniently by Elderton and Fisher.* Elderton gives the method developed by Professor Karl Pearson; Fisher gives that developed by Gram, Charlier, and Thiele.

In dealing with frequency curves and probability computations, the following definitions and notation are used here.

For any frequency distribution:

S = any value of the variable;

S' = mean value of the variable;

$$S' = \frac{\Sigma S}{n} \quad \ldots\ldots (62)$$

where n is the number of S's in the summation:

x = deviation of the variable from its mean value;

$$x = S - S' \quad \ldots\ldots (63)$$

y = frequency with which the variable occurs at the value, x, y being frequency per unit of x, the total frequency being 1.

y is also the probability that the variable will occur within the unit of x to which it corresponds.

By definition:

$$\Sigma x y = 0 \quad \ldots\ldots (64)$$

$$\Sigma y = 1 \quad \ldots\ldots (65)$$

The moment functions about the origin at S' are defined as follows:

$$\mu_0 = \frac{\Sigma y}{\Sigma y} = 1 \quad \ldots\ldots (66)$$

$$\mu_1 = \frac{\Sigma x y}{\Sigma y} = 0 \quad \ldots\ldots (67)$$

$$\mu_2 = \frac{\Sigma x^2 y}{\Sigma y} \quad \ldots\ldots (68)$$

$$\mu_3 = \frac{\Sigma x^3 y}{\Sigma y} \quad \ldots\ldots (69)$$

etc.

A more general and better measure of the spread or dispersion of the distribution than the "probable error", is the "standard deviation", which is the square root of the mean square deviation from the mean:

$$\text{Standard deviation} = \sigma = \sqrt{\mu_2} = \sqrt{\frac{\Sigma x^2 y}{\Sigma y}} \quad \ldots\ldots (70)$$

* "Frequency Curves and Correlation", by W. Palin Elderton, Lond., C. and E. Layton, 1917; "The Mathematical Theory of Probabilities", by Arne Fisher, Second Edition, New York, The Macmillan Co., 1922.

The "coefficient of variation" is the standard deviation divided by the mean value of the variable, and is of especial value in indicating the relative reliability of two materials having different average strengths:

$$C_v = \frac{\sigma}{S'} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (71)$$

The "probable error", used by the author, is only valid when the frequency curve is normal or Gaussian. It is given by:

$$r = 0.6745\,\sigma \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (72)$$

Another convenient factor is the "coefficient of skew", given by the equation:

$$C_s = \frac{\mu_3}{(\mu_2)^{\frac{3}{2}}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (73)$$

This is a rough measure of the "skewness" of the curve, the latter being defined as,

$$\frac{\text{Mean} - \text{mode}}{\text{Standard deviation}}.$$

The mode is the abscissa of the highest point of the frequency curve.

To enable the use of the best modern tables of the probability function and of the probability integral, and for ease of computation, it is convenient to express x in units of σ, or:

$$x' = \frac{x}{\sigma} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (74)$$

$$y' = \sigma\, y \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (75)$$

The probability functions are:

$$y = f(x)$$

or,

$$y' = f'(x')$$

$$p = \int_{-\infty}^{x'} f'(x')\,dx' = \int_{-\infty}^{x'} y'\,dx' = \int_{-\infty}^{x} f(x)\,dx = \int_{-\infty}^{x} y\,dx. \quad (76)$$

Values of y' and of p or $(1-p)$ for a normal or Gaussian distribution have been given,* or they may be computed, as shown by the author. For other than a normal distribution, they may be computed by the methods of Pearson or Charlier.

To obtain data having a high degree of reliability for such variables as the strength of materials requires a large number of tests. It is most unfortunate that the required frequency distributions are not more readily available. The accompanying data indicate that the author's values of r in Table 10 are derived from too limited a number of tests on columns too closely similar, and show a reliability somewhat greater than can be depended on in practical work. The accompanying tables present in probability form the following data:

* Pearson's "Tables for Statisticians and Biometricians"; Fisher's "The Mathematical Theory of Probabilities"; and elsewhere.

Table 18 gives 76 tests on the ultimate strength of structural steel in tension.

Table 19 gives 76 tests on the useful limit point of structural steel in tension.

Table 20 gives 249 tests on the ultimate strength of structural steel columns.

Table 21 gives 183 tests on the useful limit point of structural steel columns.

Table 22 gives 27 tests on the useful limit point of structural steel columns, all light sections with a ratio, $\frac{l}{r}$, of 85, forming a more homogeneous series.

Table 23 gives 73 tests on 1 : 6 concrete of different materials at an age of 28 days.

TABLE 18.—FREQUENCY DISTRIBUTION OF THE ULTIMATE STRENGTH IN TENSION OF 76 SAMPLES OF STRUCTURAL STEEL.*

Ultimate strength, in pounds per square inch.	Number of samples.	x.	Nx.	Nx^2.	Nx^3.
53 000-54 000	2	−4	− 8	32	−128
54 000-55 000	2	−3	− 6	18	− 54
55 000-56 000	6	−2	−12	24	− 48
56 000-57 000	19	−1	−19	19	− 19
57 000-58 000	11	0	0	0	0
58 000-59 000	11	1	11	11	11
59 000-60 000	5	2	10	20	40
60 000-61 000	6	3	18	54	162
61 000-62 000	7	4	28	112	448
62 000-63 000	2	5	10	50	250
63 000-64 000	2	6	12	72	432
64 000-65 000	2	7	14	98	686
65 000-66 000	1	8	8	64	512
	76	....	66	574	2 292

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1613.

$$m_0 = \frac{76}{76} = 1.0000 \qquad \mu_2 = m_2 - m_1^2 = 6.7985$$

$$\sigma = \sqrt{\mu_2} = 2.6074$$

$$m_1 = \frac{66}{76} = 0.8684 \qquad \text{Mean} = 57\ 500 + 868 = 58\ 368$$

$$\mu_3 = m_3 - 3\, m_1\, m_2 - m_1^3 = 9.8260$$

$$m_2 = \frac{574}{76} = 7.5526 \qquad C_v = \frac{\sigma}{S'} = \frac{2.6074}{58.368} = 0.04484$$

$$m_3 = \frac{2292}{76} = 30.1579 \qquad C_s = \frac{\mu^3}{\mu_2^{3/2}} = 0.5522$$

$$\lambda_3 = \frac{\mu_3 - m_1^3}{\sigma^3} = 0.5154$$

$$C_3 = -\frac{\lambda_3}{6} = -0.0859$$

The equation,

$$y' = \phi\,(x') = \phi_0\,(x') - 0.0859\ \phi_3\,(x')$$

where $\phi_0\,(x')$ is the Gaussian value of y' and $\phi_3\,(x')$ is its third derivative.

TABLE 19.—FREQUENCY DISTRIBUTION OF THE USEFUL LIMIT POINT IN TENSION OF 76 SAMPLES OF STRUCTURAL STEEL.*

Useful limit point, in pounds per square inch.	Number of samples.	Useful limit point, in pounds per square inch.	Number of samples.	Useful limit point, in pounds per square inch.	Number of samples.
17 000–18 000	1	25 000–26 000	3	33 000–34 000	12
18 000–19 000	2	26 000–27 000	4	34 000–35 000	1
19 000–20 000	0	27 000–28 000	5	35 000–36 000	4
20 000–21 000	1	28 000–29 000	4	36 000–37 000	3
21 000–22 000	1	29 000–30 000	4	37 000–38 000	4
22 000–23 000	0	30 000–31 000	5	38 000–39 000	1
23 000–24 000	6	31 000–32 000	6	39 000–40 000	1
24 000–25 000	1	32 000–33 000	6	40 000–41 000	1

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1613.

Mean = 30 300; standard deviation, σ = 5 018; C_v = 0.1655.

TABLE 20.—FREQUENCY DISTRIBUTION OF THE ULTIMATE STRENGTH OF 249 STRUCTURAL STEEL COLUMNS.*

Ultimate strength, in pounds per square inch.	Number of samples.	Ultimate strength, in pounds per square inch.	Number of samples.	Ultimate strength, in pounds per square inch.	Number of samples.
21 000–22 000	3	30 000–31 000	25	39 000–40 000	..
22 000–23 000	3	31 000–32 000	24	40 000–41 000	..
23 000–24 000	8	32 000–33 000	27	41 000–42 000	..
24 000–25 000	9	33 000–34 000	18	42 000–43 000	..
25 000–26 000	13	34 000–35 000	12	43 000–44 000	2
26 000–27 000	13	35 000–36 000	6	44 000–45 000	3
27 000–28 000	16	36 000–37 000	10	45 000–46 000	1
28 000–29 000	31	37 000–38 000	..	.....	...
29 000–30 000	14	38 000–39 000	11	.....	...
Total	....	.....	....	.....	249

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1613.

Mean = 30 607; standard deviation, σ = 4 396; C_v = 0.1436.

TABLE 21.—FREQUENCY DISTRIBUTION OF THE USEFUL LIMIT POINT OF 183 STRUCTURAL STEEL COLUMNS.*

Useful limit point, in pounds per square inch.	Number of samples.	Useful limit point, in pounds per square inch.	Number of samples.	Useful limit point, in pounds per square inch.	Number of samples.
18 000–19 000	1	24 000–25 000	20	30 000–31 000	15
19 000–20 000	4	25 000–26 000	19	31 000–32 000	8
20 000–21 000	3	26 000–27 000	27	32 000–33 000	5
21 000–22 000	8	27 000–28 000	13	33 000–34 000	3
22 000–23 000	9	28 000–29 000	15	34 000–35 000	3
23 000–24 000	17	29 000–30 000	12	35 000–36 000	1
Total	....	.....	....	.....	183

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1613.

Mean = 26 576; standard deviation, σ = 3 452; C_v = 0.1299.

TABLE 22.—FREQUENCY DISTRIBUTION OF THE USEFUL LIMIT POINTS OF 27 STRUCTURAL STEEL COLUMNS, ALL LIGHT SECTIONS, $\frac{l}{r} = 85$.*

Useful limit point, in pounds per square inch.	Number of samples.	Useful limit point, in pounds per square inch.	Number of samples.	Useful limit point, in pounds per square inch.	Number of samples.
23 000-24 000	1	27 000-28 000	3	31 000-32 000	4
24 000-25 000	1	28 000-29 000	4	32 000-33 000	2
25 000-26 000	..	29 000-30 000	3	33 000-34 000	..
26 000-27 000	4	30 000-31 000	4	34 000-35 000	1
Total	.....	.....	.....	.....	27

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1613.
Mean = 28 952; standard deviation, $\sigma = 2\,473$; $C_v = 0.0854$.

TABLE 23.—FREQUENCY DISTRIBUTION OF THE COMPRESSIVE STRENGTH, AT 28 DAYS, OF 73 SAMPLES OF 1:6 CONCRETE OF VARIOUS MATERIALS, FROM VARIOUS SOURCES, NOT INCLUDING THE AUTHOR'S DATA.

Strength, in pounds per square inch.	Number of samples.	Strength, in pounds per square inch.	Number of samples.	Strength, in pounds per square inch.	Number of samples.
600-700	1	1 800-1 900	3	2 700-2 800	2
.....	..	1 900-2 000	4	2 800-2 900	1
1 000-1 100	3	2 000-2 100	7	2 900-3 000	4
.....	..	2 100-2 200	2	3 000-3 100	1
1 300-1 400	1	2 200-2 300	5	3 100-3 200	1
1 400-1 500	2	2 300-2 400	11	3 200-3 300	..
1 500-1 600	2	2 400-2 500	7	3 300-3 400	1
1 600-1 700	1	2 500-2 600	5	.....	..
1 700-1 800	4	2 600-2 700	5	3 700-3 800	1
Total.....	....	.....	....	.....	73

Mean = 2 236; standard deviation, $\sigma = 547$; $C_v = 0.2446$; $C_s = -0.300$.

TABLE 24.—FREQUENCY DISTRIBUTION OF THE COMPRESSIVE STRENGTH, AT 28 DAYS OF 45 SAMPLES OF 1 : 9 CONCRETE OF VARIOUS MATERIALS, FROM VARIOUS SOURCES.

Strength, in pounds per square inch.	Number of samples.	Strength, in pounds per square inch.	Number of samples.	Strength, in pounds per square inch.	Number of samples.
400- 500	1	1 400-1 500	1	2 100-2 200	4
700- 800	2	1 500-1 600	2	2 200-2 300	4
800- 900	1	1 600-1 700	3	2 300-2 400	3
1 000-1 100	3	1 700-1 800	2	2 400-2 500	1
1 100-1 200	3	1 800-1 900	3	2 500-2 600	1
1 200-1 300	1	1 900-2 000	3	2 800-2 900	1
1 300-1 400	3	2 000-2 100	3	.....	..
Total	....	.....	....	.....	45

Mean = 1 728; standard deviation, $\sigma = 546$; $C_v = 0.316$; $C_s = -0.324$.

TABLE 25.—FREQUENCY DISTRIBUTION OF THE COMPRESSIVE STRENGTH OF 108 TEST CYLINDERS OF CONCRETE GIVEN BY THE AUTHOR, OMITTING THOSE WITH A STRENGTH OF MORE THAN 4 000 LB. PER SQ. IN., WHICH APPEAR TO BELONG TO CONCRETE OF AN ENTIRELY DIFFERENT CLASS.

Strength, in pounds per square inch.	Number of samples.	Strength, in pounds per square inch.	Number of samples.	Strength, in pounds per square inch.	Number of samples.
600- 700	1	1 800-1 900	7	3 000-3 100	3
700- 800	1	1 900-2 000	5	3 100-3 200	1
800- 900	....	2 000-2 100	8	3 200-3 300	2
900-1 000	1	2 100-2 200	10	3 300-3 400	1
1 000-1 100	....	2 200-2 300	5	3 400-3 500	....
1 100-1 200	4	2 300-2 400	5	3 500-3 600	1
1 200-1 300	2	2 400-2 500	3	3 600-3 700	....
1 300-1 400	4	2 500-2 600	4	3 700-3 800	2
1 400-1 500	6	2 600-2 700	5	3 800-3 900	....
1 500-1 600	4	2 700-2 800	4	3 900-4 000	1
1 600-1 700	3	2 800-2 900	6	.....	....
1 700-1 800	9	2 900-3 000	....	.....	....
Total.....	....	.....	....	.....	108

Mean = 2 031; standard deviation, $\sigma = 650$; $C_v = 0.320$; $C_s = +0.386$.

TABLE 26.—FREQUENCY DISTRIBUTION OF THE STRENGTH OF 116 CONCRETE SAMPLES GIVEN BY THE AUTHOR. ARRANGED AS PERCENTAGES OF THE MEAN, FOR EACH GROUP OF TESTS, TO OBTAIN THE RELIABILITY TO BE EXPECTED IN A HOMOGENEOUS SERIES.

Percentage of variation.	Number of samples.	Percentage of variation.	Number of samples.	Percentage of variation.	Number of samples.
−35.2	1	− 6 to − 8	6	12 to 14	3
−31.8	1	− 4 to − 6	10	14 to 16	2
−29.4	2	− 2 to − 4	12	16 to 18	3
−27.7	1	0 to − 2	12	20.4	1
−20.4	1	0 to 2	11	29.4	1
−16 to −18	1	2 to 4	9	37.2	1
−14 to −16	3	4 to 6	9	41.6	1
−12 to −14	2	6 to 8	3	45.0	1
−10 to −12	1	8 to 10	7	.....	....
− 8 to −10	8	10 to 12	3	.....	....
Total	....	.....	....	.....	116

Mean = 0; standard deviation, $\sigma = 12.26\%$; $C_v = 0.1226$; $C_s = +0.380$.

Table 24 gives 45 tests on 1:9 concrete of different materials at an age of 28 days.

Table 25 gives 108 tests on the strength of concrete test cylinders from the author's data.

Table 26 gives 116 tests on the strength of concrete, from the author's data.

For each series, the frequency distribution is given in tabular form, and the computation of σ, C_v, C_s, and the equation of a skew curve (after Fisher)

is shown in detail for Series 1 (Table 18). Series 1 and 4 are shown graphically in Figs. 14 and 15. Fig. 14 is a markedly skew distribution, and Fig. 15 is normal or Gaussian. There is as yet no decisive evidence for using other than the normal form for the frequency curves of concrete or for the useful limit point of steel. The skewness in the ultimate strength of steel in tension is probably due to the selective effect of laboratory control of manufacture, which eliminates the unusually low specimens. The absence of such a skewness, in the tests for the useful limit point, indicates that in making steel too great emphasis is placed on the ultimate strength of the product, and not enough on the more important elastic limit. A comparison of the data from these series of tests with those of the author is given in Table 27.

TABLE 27.—Comparison of the Frequency Data on Strength of Materials.

Item No.	Material.	Mean.	Coefficient of variation, C_v.	Coefficient of skew, C_s.
1	Steel—Ultimate strength in tension. (See Table 18.)	58 368	0.04484	+0.5522
2	Steel—Useful limit point in tension. (See Table 19.)	30 300	0.1655	.....
3	Steel columns—Ultimate strength. (See Table 20.)	30 607	0.1436	.....
4	Steel columns—Useful limit point. (See Table 21.)	26 576	0.1299	.....
5	Steel columns, homogeneous series—Useful limit point. (See Table 22.)	28 952	0.0854	.....
6	1:6 concrete, 28-day strength, of samples of various materials. (See Table 23.)	2 236	0.2446	−0.300
7	1:6 concrete, 28 day strength, of the same samples as above, reduced to a homogeneous series by computing standard deviation about the means of each set of tests	2 236	0.1363	.....
8	1:9 concrete, 28-day strength, of samples of various materials. (See Table 24.)	1 728	0.316	−0.324
9	Concrete—Non-homogeneous series from the author's paper. (See Table 25.)	2 031	0.320	+0.386
10	Concrete—Homogeneous series derived from above (See Table 26.)	2 031	0.1226	+0.380
11	Structural-steel columns. (See Table 10.)	.....	0.0304	.....
12	von Emperger type columns. (See Table 10.)	.....	0.0350	.....
13	Structural steel and concrete columns. (See Table 10.)	.....	0.0638	.....
14	Concrete column rod reinforcement. See Table 10.)	.....	0.0831	.....
15	Concrete column spiral reinforcement. (See Table 10.)	.....	0.0834	.....
16	Concrete column rods and spiral reinforcement (See Table 10.)	.....	0.1036	.....
17	Concrete column unreinforced. (See Table 10.)	.....	0.0865	.....
18	Cast iron	70 000	0.1780	.....

It is to be hoped that engineers connected with testing laboratories will be able to add more complete and conclusive data.

The difference in reliability between concrete specified by proportions only, and that controlled by test, is well exemplified in Table 27. It would appear to be consistent to use this difference in probability or reliability coefficients in computing the allowable working stress for the two classes, rather than the arbitrary factor, 0.6, proposed by the author.

Having determined the probability coefficients for the material under consideration, from the best data available, it remains to use these factors in a logical derivation of the factor of safety. After applying a factor of 0.50 (for concrete) to cover the effect of fatigue, the author arbitrarily chooses a

stress which will make the probability of failure 0.0000001. It is difficult to justify such a choice of the proper stress without analysis.

It is evident that the choice of the proper unit stress is an economic one. If it cost nothing to assume a low stress there would be justification in providing structures that would not fail once in a hundred million or in a hundred billion times, rather than in the ten million assumed by the author. This is not the case, however, and frequently his factor will be unjustifiably conservative.

The computation of the proper stress for the simple case where the load can be estimated definitely with negligible variation, and where the additional dead load stress caused by a decreased unit stress may be neglected, is as follows:

Let,

S' = the mean unit strength of the material at the point where structure failure takes place. The author points out that this should be 50% of the ultimate strength in the case of concrete.

C = the part of the cost of the structure which varies with the unit stress adopted.

S = the unit stress.

$C = \frac{K}{S}$, where K is a constant.....(77)

p = the probability of failure.

D = the value of the damage or loss in case of failure.

The probable loss from failure is $D\,p$. This may be considered as the cost of insurance against failure, a premium which will compensate adequately for the risk of loss and damage.

The total cost of the structure is $K' + C = K' + \frac{K}{S}$, in which K' is a constant.

For maximum economy,

$$K' + \frac{K}{S} + D\,p \text{ must be a minimum} \qquad (78)$$

From Equation (63), $S = S' + x$,

$$K' + \frac{K}{(S' + x)} + D\,p \text{ must be a minimum} \qquad (79)$$

Placing the first derivative of Equation (79) at zero:

$$\frac{-K}{(S' + x)^2} + D\,\frac{d}{d\,x}p = 0 \qquad (80)$$

But, from Equation (76), $\frac{d}{d\,x}p = y$,

$$\frac{-K}{(S' + x)^2} + D\,y = 0 \qquad (81)$$

Substituting from Equation (77), and re-arranging,

$$\frac{D}{C} = \frac{1}{S\,y} \qquad (82)$$

which may be solved for S if $\frac{D}{C}$ and the equation of $y = f(x)$ are known. $\frac{D}{C}$ is the ratio of the value of the total loss in case of failure to the part of the cost of the structure which is dependent on the unit stress adopted.

For the case when the dead load stress is not negligible, with notation as before:

Let,

L = the total live load; and

$\frac{M}{S}$ = the total dead load, where M is a constant.

The total cost of the structure:

$$C + K' = \frac{K}{S}\left(L + \frac{M}{S}\right) + K' = \frac{K\,L}{S} + \frac{K\,M}{S^2} + K' \ldots\ldots (83)$$

As before,

$$\frac{d}{d\,x}\left(\frac{K\,L}{S' + x} + \frac{K\,M}{(S' + x)^2} + D\,p\right) = 0 \ldots\ldots\ldots\ldots (84)$$

$$\frac{-K\,L}{(S' + x)^2} - \frac{2\,K\,M}{(S' + x)^3} + D\,y = 0 \ldots\ldots\ldots\ldots (85)$$

$$D\,y = \frac{K}{S^2}\left(L + \frac{M}{S}\right) + \frac{K\,M}{S^3} \ldots\ldots\ldots\ldots (86)$$

$$D\,y = \frac{C}{S} + \frac{K\,M}{S^3} \ldots\ldots\ldots\ldots (87)$$

$$\frac{D}{C} = \frac{1}{S\,y} + \frac{K\,M}{C\,S^3\,y} \ldots\ldots\ldots\ldots (88)$$

For this case, S is always larger than with Equation (82).

As an example of the application of Equation (82), take a concrete for which tests are available showing an average ultimate strength of 2 500 lb. per sq. in., with a C_v of 12.5 per cent. Using the author's factor of 0.50 for repeated loads, $S' = 1\,250$ lb. per sq. in., and $\sigma = 156$ lb. per sq. in. The computation is given in Table 28.

TABLE 28.—COMPUTATION OF $\frac{D}{C}$ FOR $S' = 1\,250$, $\sigma = 156$.

S, stress, in pounds per square inch.	x $= S - 1\,250$	x' $= \frac{x}{156}$	y', from probability tables.	y $= \frac{y'}{156}$	$\frac{D}{C}$ $= \frac{1}{Sy}$
300	−950	−6.09	0.000000004	0.000000000026	128 000 000
400	−850	−5.45	0.0000001416	0.000000000907	2 760 000
500	−750	−4.81	0.000003776	0.0000000242	82 700
600	−650	−4.17	0.0000668	0.000000428	3 900
700	−550	−3.53	0.0007853	0.00000503	284
800	−450	−2.88	0.006307	0.0000404	30.95
900	−350	−2.24	0.08246	0.000208	5.34
1 000	−250	−1.60	0.1109	0.000706	1.416

The usual range of $\frac{D}{C}$ is from 10 to 10 000, giving working stresses from 865 to 565 lb. per sq. in. The author's factor of $p = 0.0000001$ gives $x' = 5.20$, a working stress of 440 lb. per sq. in., and implies for this case a value of $\frac{D}{C}$ of 670 000.

The values of S are plotted as abscissas and those of $\frac{D}{C}$ as ordinates in the upper curve of Fig. 16.

A similar computation for a 1:6 concrete, specified by proportions only, with the data of Item 7, Table 27, gives the lower curve in Fig. 16. The markedly lower stresses permissible for this case are a striking argument for adequate laboratory control. Thus, for a $\frac{D}{C}$ of 1 000, the allowable stress is only 30.3% of that for the tested concrete, and for a $\frac{D}{C}$ of 10, only 64.2 per cent. It is to be noted that the manner in which the tests were compiled, from a wide variety of sources, gives a greater degree of heterogeneity—and hence of dispersion or unreliability—than will normally need to be provided against in practice, even without laboratory control. The writer believes that, until additional evidence is forthcoming, a coefficient of variation of 18% is a reasonable estimate for commercial concrete without laboratory control. The curve for such a concrete, with an ultimate strength of 2 500 lb. per sq. in., and a time load or repeated load factor of 50%, is shown by the middle curve in Fig. 16.

The proper value of $\frac{D}{C}$, in theory, should be determined for each individual case of design. It is absurd to assume that the same working stress is economical, and should be used, in the design of an isolated water tank, where the only damage in case of failure would be the loss of the structure, with a $\frac{D}{C}$ of from 2 to 5, and in the design of a metropolitan bridge carrying heavy traffic at all times, where failure would entail a heavy loss of life and very expensive dislocation of traffic.

It is suggested that, for an average building or bridge, $\frac{D}{C}$ be taken at about 1 000; for storage warehouses not intended for human occupation, $\frac{D}{C}$ should be taken at about 100; for bins and tanks holding cheap materials, for electric poles, etc., $\frac{D}{C}$ should be taken at about 10; and for fence posts and similar work, $\frac{D}{C}$ should be taken at about 2. In any important case of design, $\frac{D}{C}$ should be estimated carefully for the special conditions which govern. This

makes it possible to choose a proper unit stress for design, which may then be modified by the factors for $\frac{l}{r}$ and the type and quantity of reinforcement suggested by the author.

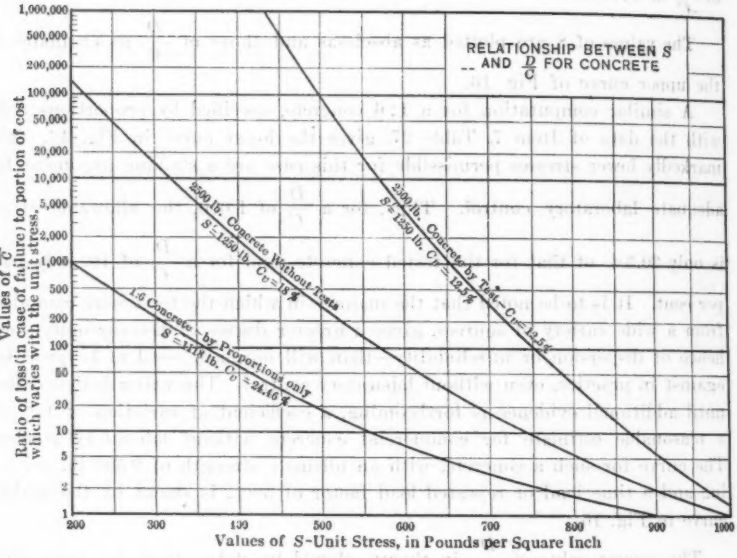


FIG. 16.

The author is to be congratulated on his recognition, and thanked for his presentation, of the possibilities of a statistical study of the reliability of materials, and for the application of probability mathematics to this important subject. The methods of probability furnish a definite, sound, and logical basis for the choice of unit stresses in design, and should be used widely and carefully. The great need is for more adequate data on the frequency distributions of strength for the various materials of construction. The routine of recording tests should be arranged with this necessity clearly in mind. No amount of committee work, or averaging of the empirical guesses of different men, can produce results as trustworthy as those which may be obtained by a small increase in the numbers of test samples, and analysis by the probability method outlined.

The writer believes that the two concepts of engineering design, as being primarily an economic problem, and as dealing with variable elements for which definite and fixed predictions cannot be made, but for which a close estimate of probabilities can be formed, are fundamental. Advance in the theory and in the practical details of engineering design can only take place if these fundamental concepts are deeply ingrained in all processes of thought,

kept clearly in mind, and applied consistently in the technical details of computations.

L. J. MENSCH,* M. AM. SOC. C. E. (by letter).—The author is to be congratulated on the clear and forceful manner in which he has presented his subject in Section I; he established, as his foundation, tests made on concrete columns in the United States, and succeeded in developing, for the strength of such columns, empirical formulas more convenient and more accurate than the rules given generally in building codes or in the recommendations of the Joint Committee on Concrete and Reinforced Concrete.

The subject, however, needs much more consideration than the author has given it. He has neglected a great many tests made in America and the fundamental tests made by European authorities, namely, about 50 columns tests by Considère and the French Committee;† 170 tests by the Austrian Committee;‡ a very large number of tests by Professors V. Bach,§ M. Rudeloff,‖ Morsch,¶ etc.

As these tests are not well known in America, the writer presents some of them in Tables 29 and 30. A study of these tests would enable the author to correct and supplement many of his statements. For example, for spirally reinforced concrete columns, the author has stated the following three important principles:

(1) The vertical reinforcement is stressed higher in a spirally reinforced concrete column than in an ordinary concrete column; a known fact, which has been stated by very few writers, but which is recognized in a ruling of the Chicago Building Department;
(2) The effect of the spiral reinforcement is proportional to the strength of the plain concrete in the core;
(3) The effect of the spiral reinforcement is independent of the elastic limit of the material of the spiral, as long as the limit of the latter is greater than 36 000 lb. per sq. in.

All who are familiar with the history of reinforced concrete, know that Considère, the inventor of the spirally reinforced concrete column, derived the idea of the spiral reinforcement from the behavior of sand in sand-boxes. Although sand has no compressive stress in an unconfined state, it has great supporting power if confined laterally, and, from the theory of pressure in granular materials, he derived the theorem that the relation of vertical to lateral pressure equals 2.4. Later researches on this relation for sand have shown that it is even greater than 3. Considère maintained that for concrete this relation should not be less than that for plain sand, and assumed that the spiral reinforcement has to carry only $\frac{1}{2.4}$ times the increase of the strength of the core due to the spiral, a view adopted in many building codes.

* Gen. Contr., Chicago, Ill.

† Commission du Béton Armé, Paris, 1907.

‡ Mitteilungen über Versuche ausgefuhrt vom Eisenbeton-Ausschuss de Oesterreichischen Ingenieur und Architekten-Vereins, Heft 3, 1912.

§ *Zeitschrift* des Vereines deutcher Ingenieure, 1915, p. 898.

‖ *Deutscher Ausschuss für Eisenbeton*, Heft 5, 21, 28.

¶ *Der Eisenbeton Bau*, Band I, 1920.

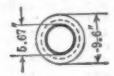
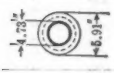
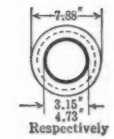
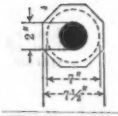
TABLE 29.—TESTS OF

No. of test.	Length, in feet.	Core diameter, in inches.	Area, in square inches.	Vertical Steel. No. of rods.	Diameter of rods, in inches.	Area of vertical reinforcement, in square inches.	Percentage.	Spiral Reinforcement. Diameter of spiral reinforcement, in inches.	Pitch, in inches.	Percentage.
										TESTS BY THE
1	6.56	7.16 sq.	51.2	Plain concrete without vertical reinforcement.				No spiral reinforcement used.		
1 *a*	"	"	"							
2	"	"	"	8	0.59	2.19	4.26			
2 *a*	"	"	"	"	"	"	"			
3	"	7.25 round	41.2	6	0.394	0.73	1.42	0.236	0.394	4.92
3 *a*	"	"	"	"	"	"	"	"	"	"
4	"	"	"	"	"	"	"	"	0.55	3.49
4 *a*	"	"	"	"	"	"	"	"	"	"
5	"	"	"	"	"	"	"	0.315	0.985	3.46
5 *a*	"	"	"	"	"	"	"	"	"	"
6	8.20	"	"	6	0.355	0.592	1.15	0.236	0.748	2.58
6 *a*	"	"	"	"	"	"	"	"	"	"
7	8.50	"	"	6	0.314	0.469	0.91	"	0.943	2.05
7 *a*	"	"	"	"	"	"	"	"	"	"
8	9.81	"	"	6	0.276	0.358	0.69	"	1.18	1.64
8 *a*	"	"	"	"	"	"	"	"	"	"
9	13.1	7.16 sq.	51.2	Plain concrete without vertical reinforcement.				No spiral reinforcement used.		
9 *a*	"	"	"							
10	"	"	"	8	0.355	0.79	1.53			
10 *a*	"	"	"	"	"	"	"			
11	"	7.25 rd.	41.2	6	0.314	0.467	0 91	0.236	1.77	1.09
11 *a*	"	"	"	"	"	"	"	"	"	"
12	"	"	"	"	"	"	"	0.196	1.18	1.13
12 *a*	"	"	"	"	"	"	"	"	"	"
13		5.5 rd.	23.8					0.236	1.18	2.7
14		"	"	No vertical steel.				0.157	0.59	2.52
15		"	"					0.236	1.07	3.0
16		29.4 sq.						No spiral used.		
										TESTS BY
17	1.67	11	95.0	8	0.395	1.00	1.11	0.315	1.97	1.45
18									1.57	1.82
19									1.26	2.25
20									0.95	3.05
21									0.63	4.45
22									0.315	9.10
23									1.26	2.1
24									0.95	2.65
25									0.63	4.15
26									0.315	8.10
										TESTS BY PROFESSOR
73	4.25	10.75	88.5	8	0.552	1.925	2.25	0.286	1.18	2.06
74										
75										
76										
77	"	"	"	"	"	"	"	"	"	"
78										
79	"	"	"	"	"	"	"	"	"	"

CONCRETE COLUMNS.

TOTAL LOAD AT FAILURE.		Load carried by concrete. Ultimate strength, in pounds per square inch, corresponding to that obtained in plain concrete.	INCREASE OF LOAD.				Mr. Tucker's coefficient, k.	Test cylinders or columns, stress, in pounds per square inch.	Age of concrete.
Over entire section, in thousands of pounds.	In pounds per square inch.		Due to vertical steel.	Due to spiral reinforcement.	Per square inch of core area, due to spiral reinforcement.	Due to 1% of spiral per square inch of core area.			
FRENCH COMMITTEE.									
176.4	3 450	176 400	0	.....	.....	.....	...	3 450	5 months
134.5	2 620	134 500	0	.....	.....	.....	...	2 620	"
238	4 600	176 400	61 600	.....	.....	.....	...	3 450	"
210	4 040	134 500	69 500	.....	.....	.....	...	2 620	"
375	9 100	142 000	25 600	207 400	5 050	1 025	30	3 450	"
345	8 360	108 000	"	211 600	5 400	1 100	42	2 620	"
363	8 780	142 000	"	195 400	4 740	1 360	39	3 450	"
335	8 120	108 000	"	201 600	4 900	1 400	53	2 620	"
315	7 680	142 000	"	147 400	3 580	1 035	30	3 450	"
325	7 830	108 000	"	192 600	4 660	1 345	52	2 620	"
325	7 830	142 000	20 700	162 300	3 930	1 520	44	3 450	"
300	7 300	108 000	"	172 350	4 180	1 620	61	2 620	"
281	6 800	142 000	16 400	122 600	2 970	1 450	43	3 450	"
247	6 000	108 000	"	122 600	2 970	1 450	56	2 620	"
230	5 340	142 000	12 500	65 500	1 580	965	28	3 450	"
218	5 280	108 000	"	97 500	2 360	1 440	55	2 620	"
144.5	2 820	144 500	0	.....	.....	.....	...	2 820	"
107	2 160	107 000	0	.....	.....	.....	...	2 160	"
161	3 140	144 500	16 500	{ or an average of 32 750 lb. per sq. in. of vertical reinforcement				2 820	"
156	3 040	107 000	49 000					2 160	"
224	5 410	116 000	15 300	92 700	2 250	2 060	73	2 820	"
195	4 720	89 000	"	90 700	2 210	2 030	94	2 160	"
210	5 100	116 000	"	78 700	1 920	1 700	60.4	2 820	"
200	4 850	89 000	"	95 700	2 330	2 060	95	2 160	"
154	5 800	28 100	0	125 900	5 280	1 955	166	1 180	.....
159	6 050	28 100	0	130 900	5 490	2 030	172	1 180	.....
148	5 620	28 100	0	119 900	5 000	1 850	157	1 180	.....
34.2	1 180	34 200	0	0	0	0	.....	.....	.....
PROFESSOR BACH.									
413	4 350	176 000	48 000	189 000	1 990	1 370	74	1 850	28 days
452	4 760	"	"	228 000	2 400	1 320	71	"	"
520	5 470	"	"	296 000	3 120	1 390	75	"	"
605	6 370	"	"	381 000	4 020	1 375	74	"	"
770	8 100	"	"	546 000	5 750	1 290	70	"	"
1 220	12 940	"	"	1 006 000	10 520	1 150	62	"	"
357	3 760	83 000	48 000	226 000	2 380	1 135	130	875	"
403	4 250	"	"	272 000	2 860	1 080	123	"	"
578	6 080	"	"	447 000	4 700	1 150	131	"	"
978	10 300	"	"	847 000	8 900	1 100	126	"	"
M. RUDELOFF.									Elastic limit of spirals.
452.5	5 100	214 000	86 000	152 500	1 710	830	34.5	2 410	44 800
590	6 660	"	"	290 000	3 280	1 590	65.8	"	85 600
654	7 400	206 000	"	362 000	4 090	1 980	85.4	"	100 000

TABLE 30.—TESTS OF VON EMPERGER COLUMNS,

	Column No.	Outside diameter, in inches.	Core diameter, in inches.	Actual area of concrete within core, in square inches.	Vertical Reinforcement. Number of bars.	Vertical Reinforcement. Diameter of bars, in inches.	Vertical Reinforcement. Area of bars, in square inches.	Spiral Reinforcement. Diameter of reinforcement, in inches	Spiral Reinforcement. Pitch, in inches.	Spiral Reinforcement. Equivalent area, in square inches.	Spiral Reinforcement. Percentage of full core.
Tests by Austrian Concrete Institute (See Heft 3											
	I°	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
	II° *	11.4	9.6	47.14	8	0.197 rd.	0.248	0.394	1.57	2.37	5.21
	III° *	"	"	"	"	"	"	0.276	0.788	2.28	3.1
	8	5.91	5.91	9.86	6	0.197 rd.	0.183	0.15 rd.	3.15	1.03	3.77
	9	"	"	"	"	"	"	"	"	"	"
Tests by Professor Kirch at the											
	12	7.88	7.88	40.98	6	0.197 rd.	0.183	0.15 rd.	3.15	1.375	2.81
	13	"	"	"	"	"	"	"	"	"	"
	14	"	"	"	"	"	"	"	"	"	"
	15	"	"	"	"	"	"	"	"	"	"
	16†	"	"	"	"	"	"	"	"	"	"
	17	7.88	7.88	31.20	"	"	"	0.78 rd	1.57	0.61	1.25
	18	"	"	"	"	"	"	0.15 rd.	3.15	1.375	2.81
	19	"	"	"	"	"	"	0.181 rd.	3.15	2.02	4.13
	20	"	"	"	"	"	"	0.15 rd.	3.15	1.375	2.81
von Emperger's Tests,											
	3 *b*	15.75	14.95	15.7	8	0.394 rd.	0.97	Ties	.....	1.24	0.69
	4 *b*	"	"	"	"	"	"	"	.....	"	"
	5 *b*	"	"	"	"	"	"	Spiral	.....	"	"
	10 *b*	"	"	"	"	"	"	Ties	.....	"	"
	6 *b* ‡	"	"	"	"	"	"	"	.....	"	"
	7 *b*	"	"	"	"	"	"	"	.....	"	"
	8 *b*	"	"	"	"	"	"	"	.....	"	"
Tests by Professor C. F. Gebhard, in 1918, at Armour Institute											
	1	7½	7	38.5	8	0.25 rd.	0.39	No. 12	.....	.....	1.0
	2	.....	2	"	0	0		.....	.....	0.39	
	3	.....	"	.....	0	0		Cast	iron	only.	
	4	.....	.....	.....	0	0					
	5	7½	7	38.5	8	0.25 rd.	0.39	No. 12	.....	0.39	1.0
	6	"	"	"	"	"	"	"	"	"	"

* Column ends of Nos. II° and III° enlarged to 16 in.

† This column had two joints at quarter-points in cast-iron column, and was tested with an

‡ Cinder concrete.

IN 1911, 1912, AND 1916, 1 : 2 : 4 CONCRETE.

Cast Iron.				Length of column, in inches.	Maximum load, in pounds.	Load considered carried by concrete.		Load considered carried by cast iron.		Load at first crack, in pounds.	Test cylinder.
Outside diameter, in inches.	Thickness, in inches.	Area, in square inches.	Percentage of actual concrete area.			Pounds per square inch.	Total pounds.	Total pounds.	Pounds per square inch.		
(1911), PUBLICATION OF THAT INSTITUTE.)											
5.67	0.375	6.24	.....	120	307 000	.....	.....	307 000	49 200	.....	.....
"	"	"	13.2	"	706 000	7 000	330 000	373 000	59 700	.....	.....
3.21	"	"	"	"	765 000	8 000	378 000	387 000	62 000	.....	.....
3.1											
3.77	0.2165	3.12	31.6	35.6	392 000	8 000	79 000	313 000	100 000	247 000	.....
"	"	"	"	53.2	356 000	7 000	69 000	287 000	92 000	269 000	.....
POLYTECHNIC SCHOOL, VIENNA, 1912.											
3.15	0.3465	3.06	6.25	35.5	659 000	8 000	328 000	331 000	108 000	266 000	.....
"	"	"	"	53.2	513 000	6 000	246 000	267 000	87 300	203 000	.....
"	"	"	"	71	399 000	"	246 000	153 000	50 000	315 000	.....
"	"	"	"	71	403 000	"	246 000	157 000	51 300	284 000	.....
"	"	"	"	71	397 000	"	246 000	153 000	50 000	250 000	.....
4.73	0.2165	3.12	10	35.5	589 000	6 500	203 000	386 000	116 000	274 000	.....
2.81	"	"	"	"	590 000	7 500	234 000	356 000	114 000	295 000	.....
4.13	"	"	"	"	605 000	8 000	250 000	355 000	113 600	266 000	.....
2.81	"	"	"	53.2	425 000	6 000	187 000	238 000	76 300	336 000	.....
IN DRESDEN, IN 1916.											
.....	.....	2.13	13.6	79	2 060 000	3 000	471 000	1 589 000	74 600	.....	1 840
.....	.....	"	"	"	1 985 000	"	"	1 494 000	70 200	.....	1 870
.....	.....	"	"	"	1 910 000	"	"	1 439 000	67 600	.....	1 810
.....	.....	"	"	"	2 190 000	"	"	1 729 000	80 900	.....	2 070
.....	.....	"	"	"	1 760 000	2 000	314 000	1 446 000	67 900	.....	806
.....	.....	"	"	"	2 060 000	3 000	471 000	1 589 000	74 600	.....	1 980
.....	.....	"	"	"	2 170 000	"	"	1 699 000	77 800	.....	1 970
OF TECHNOLOGY. 1 : 1 : 2 CONCRETE. COLUMNS 3 MONTHS OLD.											
No cast iron.					193 000	5 010	193 000	0	0	.....	None
								60 700	18 775	.....	.....
								57 400			
2.0	0	3.14	8.15	.....	358 300 333 100	5 010	177 300	168 200	53 500	.....	.....

eccentricity of 0.8 in. at the top.

In the case of sand or very poor concrete, confined inside spiral reinforcement, the author's coefficient k (Equation 21) would become very large. A positive proof, that the effect of the spiral reinforcement depends very little on the strength of the concrete core and is nearly constant, is given by Professor Bach's tests on columns of concrete of two mixes of widely different strength, namely, of 1850 and 875 lb. per sq. in., as given in Table 29, which tests cannot be contradicted. The actual increase of the strength of the core due to each percentage of spiral reinforcement for the richer concrete is only about 10% higher than for the poorer concrete, and the speculative value of k varies from 62 for the richer to 131 for the poorer concrete. Tables 29 and 30 show that spiral reinforcement is still very effective for percentages of more than 2 and up to 9.

Table 29 contains the only tests known to the writer, which were made for the purpose of ascertaining the effect of the elastic limit of the spiral reinforcement. These tests (Nos. 73 to 79) were made on seven columns, all having the same mix of concrete (of an ultimate strength of approximately 2 400 lb. per sq. in.) and 2% of spiral reinforcement with an elastic limit from 44 800 to 100 000 lb. per sq. in. From these tests, it is found that the increase in strength of the core, for the same percentage of spiral reinforcement and the same strength of concrete, is proportional to the elastic limit of the hooping. It should be noted that, with hard wire, the difference between the elastic limit and the ultimate strength hardly amounts to 10 per cent. The tests by Considère and the Austrian Committee also show that, for a spiral reinforcement of more than 1 or 2%, the increase of strength due to the hooping depends on the pitch of the spiral which should be very much less than is usually assumed in such cases. It appears that, under high pressure, the concrete flows through the spaces of the spiral hooping which probably acts as though the effective area of the restrained core became smaller in diameter.

In Table 29 it should be noted that the space, from center to center of hooping wire, is less than 1 in. for 3% and less than $\frac{3}{4}$ in. for 5% of spiral hooping. When conclusions are drawn from tests in which the elastic limit of the spiral is not known, or the pitch of the spiral is comparatively large, confusion arises, and authors do not agree. To sum up, if all the available tests are studied and the variables just mentioned are taken into account, it will be found that the author's coefficient, k, is not a constant; that the effect of the spiral reinforcement may rather be considered an increase of the core strength, which depends only to a slight extent on the strength of the concrete itself; and that the column is stronger, the harder the steel and the smaller the pitch of the hooping. From all available tests, the conclusion can safely be drawn that the increase of core strength averages about 1 500 lb. per sq. in. of core area for 1% spiral reinforcement, using hard wire and small spacing, which agrees very well with the formula proposed by Professor Talbot in his report on the tests by the American Concrete Institute.

The author helped to make the tests on the von Emperger columns at the Pittsburgh Laboratory of the Bureau of Standards, in 1916, and therefore, is familiar with the behavior of hooped concrete columns with cast-

iron reinforcement. His empirical formula (Equation 32) for the strength of such columns may be accepted as being correct as to type, but not as to details.

The writer cannot agree with Mr. Tucker that the hooped concrete in the von Emperger columns, which he tested, should be considered as having sustained more than 4 778 lb. per sq. in. This assumed ultimate stress is the average of the two spirally hooped columns tested with 0.69% spiral and 0.71% vertical reinforcement, without a cast-iron core, and is in very good agreement with similar tests on hooped concrete columns by other investigators. Mr. Tucker knows that one of the standard spirally reinforced columns failed at a total load of 518 500 lb., the corresponding test cylinder at 3 890 lb. per sq. in., and the other failed at 562 000 lb. and its test cylinder at only 3 370 lb. per sq. in. As the agreement between cylinder strength and column strength was not particularly good, the writer offers the explanation that the workmen were accustomed to tamping columns, but not to making test cylinders, which sometimes were made of a shovelful of concrete which was entirely too wet.

Mr. Tucker's statement that the hooped columns with cast-iron reinforcement failed at a strain of 0.0034 is a surprise to the writer, who many times has measured the ultimate strain between the platens of the testing machine as soon as the scale indicated that the maximum load was reached, often not a second intervening between the two occurrences. In no case was the ultimate strain less than 0.006, the average for all columns measured was 0.0082, and the ultimate strain at the point of failure probably exceeded 0.01 in. per in.

The von Emperger columns tested at Pittsburgh had the rather low spiral reinforcement of 0.69%, and the cast-iron reinforcement failed in the 10-ft. columns at a stress of about 42 000 lb. per sq. in. To find the effect of a higher percentage of hooping on the cast-iron reinforcement, the writer had columns made, as shown in Table 30, in July, 1918, and tested in October, 1918, by Professor C. F. Gebhard, of the Armour Institute of Technology. Although the standard hooped concrete columns with 1% of spiral reinforcement failed at 5 010 lb. per sq. in., the von Emperger columns failed at 5 010 lb. per sq. in. on the concrete core and 52 000 lb. per sq. in. on the cast-iron reinforcement.

Table 30 shows that in the von Emperger columns a still higher percentage of hooping allows the cast-iron core additional carrying capacity. The concrete inside the spiral hooping was given the highest possible credit (from 6 000 to 8 000 lb. per sq. in.) for a spiral hooping of 3 to 4 per cent. It was found that the cast-iron reinforcement contributed from 50 000 to 116 000 lb. per sq. in. at the ultimate load, depending on the quality of the cast iron, the strength of the hooped concrete surrounding the cast-iron core, and the length of the columns.

The writer joins the author in a plea to manufacturers of cast-iron pipe and columns, who in the last 10 years or more have introduced perfected methods of casting columns vertically, and who, lately, have also been using

electric ovens to melt the pig iron, and have established chemical and mechanical laboratories in their plants, to have their products tested by independent experts. Such tests would do away with the prejudice against using cast-iron columns, and might result in an economy in the construction of heavy buildings.

In Section I, the author adhered strictly to actual tests and, with the material at hand, produced excellent results. In Section II, however, he lost himself in the realms of higher theory and speculation, and this part of the paper not only contains false theory, but is very confusing.

In Section III, he presents a very interesting application of the theory of probability to the selection of the reliability factor of building materials. Probably not one engineer in a hundred has ever studied this most speculative part of higher mathematics. It would be advisable if the author, in his closure, would give the frequency curves of the actual tests, and show how the reliability factor can be found directly from these curves. In practice, this very thing is done. Where a large number of tests center around a certain value, that particular value is taken as the average test result, and the much higher and much lower values are disregarded as exceptions or real errors.

Section IV contains some very meritorious reflections on the manner in which the factor of safety or the working stresses should be ascertained. The writer is not convinced that the Joint Committee ever recommended dangerous stresses. Accepting even the author's viewpoint, although the stresses may appear to be high for concrete 28 days old, they will be low for a concrete 1 year old, and, as all practical men know, a column rarely carries the design load within a year, or even during the life of the structure.

In Part D of Section IV, the author finds that the reliability theory favors the von Emperger column too much, which, however, the supplemental tests produced by the writer in Table 30 only further confirm, and he tries to reduce the working stresses by new arguments. He forgot that, in a number of tests at Pittsburgh, Mr. P. H. Bates, Superintendent in charge of the laboratory of the U. S. Bureau of Standards, insisted that the testing machine continue to run after the maximum load on the von Emperger column had been reached; after a short time a few spiral hoopings burst, and the concrete surrounding the cast iron fell out and exposed the core for a length of probably 6 in. At this time, the load on the column indicated by the machine was about 80% of the ultimate. This load was carried by the cast iron alone for the 6-in. length just mentioned, which corresponds to a stress of about 60 000 to 70 000 lb. per sq. in. on the cast iron.

Mr. Tucker, being an adept in the theory of probabilities, will easily find that the probability that the weakest part of the hooped concrete surrounds the weakest part of the cast iron is extremely small, and to speak of allowing a stress in the core only as great as in a plain cast-iron column means to disregard all tests and to stop progress.

The use of von Emperger columns in large bridges and buildings has been described in many articles in American, French, English, and German technical journals, and in a number of booklets.

Dr. von Emperger—who is well known in America as the builder of the first Milan bridges nearly 30 years ago—has built bridges on his system up to 300 ft. span in competition with light steel and wooden bridges, and has designed a large number of buildings in which his columns have been used.

Mr. Albert Kahn, of Detroit, Mich., has used these columns in two large 12-story buildings, with column loads of 2 500 000 lb. They have also been used in Chicago, Ill., in the 10-story Surf Hotel, in the 12-story McKinlock Building, in two buildings of the National Tea Company, in the Rienzi Hotel, etc., and have been adopted for the largest apartment building project in Chicago, now in course of construction, comprising two 17-story buildings with column loads up to 1 400 000 lb. These columns have effected a saving of about $200 000 over steel skeletons in this project.

Mr. Godfrey does not believe that the von Emperger columns can be spliced properly. Tests made at the Bureau of Standards proved that the simple contact of the cast-iron core pieces is sufficient to obtain full efficiency. The fact is that columns with spliced cores failed at a higher load than the same columns without splices. von Emperger columns are used only for heavy loads; and the eccentricities of loading, from girders on one side, or from wind, are relatively smaller than in columns carrying smaller loads.

The condition that tension occurs on one side of a von Emperger column is perhaps never found. Therefore, the standard splice required in Chicago, as shown in Fig. 17, for a 25-in. column, should fulfill all requirements of good engineering design.

Mr. Godfrey believes that high-grade cast iron is necessary for the success of von Emperger columns. This is incorrect. Common grades of cast iron (often very defective) have been used in the tests at the Bureau of Standards and at the Armour Institute, and most of the buildings just mentioned had cast-iron cores made by water-pipe manufacturers from standard grades of cast iron. High-grade cast iron has been used only by Dr. von Emperger in Europe, and his tests show a strength of cast-iron cores in his columns of 100 000 lb. per sq. in. at the time of failure of the concrete column.

Like the Joint Committee on Concrete and Reinforced Concrete, the author has evaded a very difficult part of the subject, that is, eccentrically loaded columns. If a concrete column in a structure shows signs of distress, a load of appreciable eccentricity may be suspected. All outside columns of a building are eccentrically loaded, and the eccentricity of loading is very marked in the upper two or three stories. The standard textbooks treat the design of combined axial loading and bending in a very complicated manner (the solution of cubic equations being required), so that only rarely are such columns designed properly. The theory is not only difficult to apply, but is faulty, as the test results differ from it often by 50% and more.

The writer has developed a new theory for the design of eccentrically loaded compression members which may be found in his discussion* on the

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1667.

Final Report of the Committee on Steel Columns and Struts. He has compared this theory with all published tests of eccentrically loaded columns, made of masonry, steel, or reinforced concrete, and found it to be in perfect agreement with the tests.

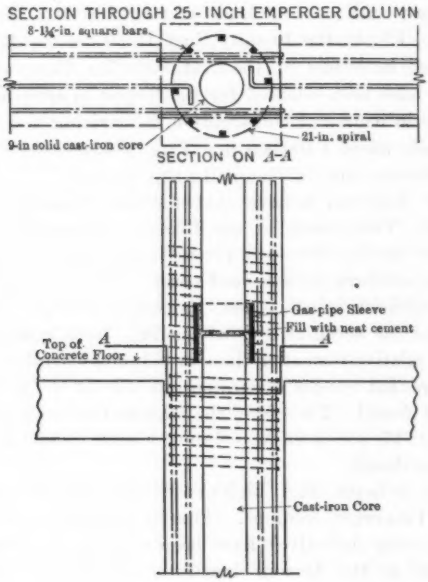


FIG. 17.

Fig. 18 shows the result of this new theory, for concrete columns with vertical steel reinforcement only, and for hooped concrete columns having a core strength on the spirally reinforced concrete alone of 5 000 lb. per sq. in., with various percentages of vertical reinforcement.

If e is the distance of the point of application of the load from the center of gravity of the section, and d is the side or diameter of the column, the strength of an eccentrically loaded column is a function of $\frac{e}{d}$, for which fraction the writer proposes the name "relative eccentricity."

The value of the diagram will be made clear by examples: A column of 3 000-lb. concrete, with a total of 2% vertical reinforcement and a relative eccentricity of $\frac{3}{10}$, can sustain just 50% of the load which would be carried by the same column if concentrically located; or, a hooped concrete column, having an ultimate strength of 5 000 lb. per sq. in., including the effect of the hooping, with 2% vertical steel, and a relative eccentricity of $\frac{3}{10}$, will carry 57.5% of the corresponding concentric load.

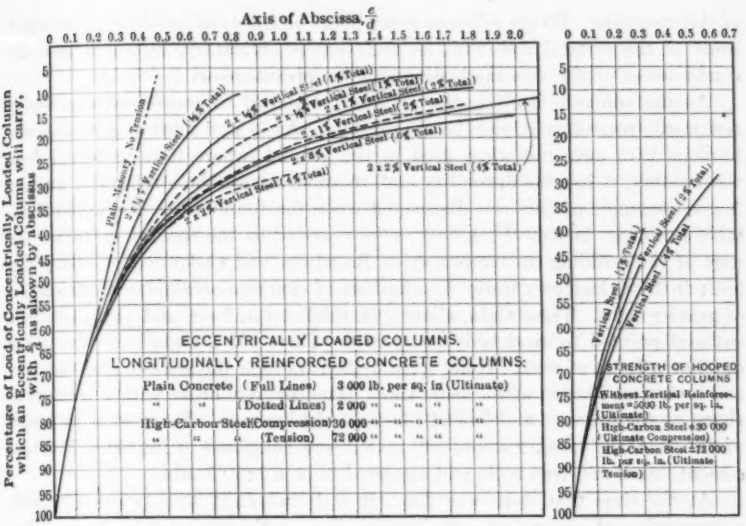


FIG. 18.

It is very remarkable that, for values of the relative eccentricity up to $\frac{2}{10}$, the loads, carried by eccentrically loaded columns, in percentage of the load of the concentrically loaded columns, is nearly the same for all types of concrete columns, and this ratio is given by the simple expression, $\frac{1}{1 + \pi \frac{e}{d}}$, in which, $\pi = 3.14$. This formula is correct for heavily reinforced concrete columns for relative eccentricities as large as $\frac{6}{10}$, which extension should include nearly all cases found in practice.

JOHN F. O'ROURKE,* M. AM. SOC. C. E.—Generally speaking, all concrete reinforcement should either be placed properly, or omitted. The bearing power of a concrete column is increased by using vertical reinforcing rods when they are stressed so that the length of compression is the same in the concrete as in the steel. As the modulus of elasticity is more constant for steel than for concrete, it is difficult to obtain this condition. As it also is difficult to determine the adhesion of the concrete to the steel, on which everything depends, the use of vertical rods in columns is of uncertain value. The "spider" reinforcement, on the contrary, acts like the hoops of a barrel. Concrete thus reinforced is free to compress vertically and the reinforcement binds it laterally, greatly increasing its strength. Some authorities ascribe more than twice the efficiency to steel used in this manner than to that used in vertical reinforcement, for which reason the speaker is inclined to favor strongly the use of "spider" reinforcement when it is not incompatible with the other parts

* Cons. Engr. and Pres., O'Rourke Eng. Const. Co., New York City.

of the structure. Where columns carry floors or fronts of buildings, into which some of the vertical rods radiate, their use is obligatory, in which case, the combination of the two methods might be advantageous.

As the concrete itself is the principal material in reinforced concrete, a few words regarding it may be appropriate. As is well known, good concrete hardens with age, and when used in the ground, where it is protected from the weather, it has the durability and, in time, almost the strength of granite. A good example of this was discovered when the foundations of the old Equitable Building, in New York City, were removed by the speaker in 1913. The part of the building on the corner of Broadway and Cedar Street, which was built in 1868, had very heavy foundations of concrete overlaid with two layers of granite slabs. These slabs, about 2 ft. thick, 10 ft. long, and 5 ft. wide, were cut and pointed to good beds and joints. These old foundations were taken out like a ledge of rock. Holes were drilled through the granite and concrete, and both were blasted at the same time. The concrete broke with a fracture as sharp as the granite, and looked like a hard bluestone. From what he knows of concrete, the speaker would say that it was composed of imported Portland cement, sand, and trap rock, and appeared to be a 1:2:5 mix.

At the time that work was done, it was customary to mix concrete with a proportion of water just sufficient to show at the top of 6-in. layers when thoroughly rammed. It would appear that the masons who did the Equitable work understood their business, because no loose aggregate was found in the concrete, through which the matrix was very evenly distributed. It required both skill and care to accomplish this. In later times, the method of mixing concrete with a surplus of water, insuring a better distribution of the matrix, was generally adopted. This wet concrete also admits of being chuted to place without separating the aggregate, which is another reason for its general use. It is the speaker's opinion that the method of mixing concrete wet so that it can be spaded or vibrated in place is the only reliable one, and that such concrete as that found at the Equitable Building, mixed by the old method, was exceptional. This statement is based on experience in taking out other concrete that was mixed in that manner, and finding sometimes a considerable proportion of it entirely without matrix and nothing but loose stone. As a matter of fact, concrete placed in moulds or forms does not usually admit of being rammed, but, when mixed wet, may be consolidated into a strong, dense mass by spading or vibration, even in the most complicated forms. The speaker approves of the spiral, or "spider", method of reinforcing concrete columns modified, when necessary, as described.

S. H. Ingberg,* Esq. (by letter).—Empirical formulas are reliable only when they are based on a sufficiently wide range of facts comparable in kind with those to which the formulas are applied. The formulas deduced in this paper for column strength and reduction of strength with length are based on laboratory tests conducted during a period of 15 years, none of which is more recent than 1914. During and subsequent to this period, there have been many changes and developments in methods of manufacture and placing concrete, and in

* Physicist, U. S. Bureau of Standards, Washington, D. C.

determining the difference between field-made and laboratory-made products. That formulas based on these tests can apply with much precision to the field-made concrete of to-day, is open to question. They may be taken with caution to give information on the effect of amount and type of reinforcement, concerning which further comments will be made under the different column types.

Length-Strength Relations.—In deducing empirical formulas for reduction of strength due to length, the author calls attention to the reduction with length that should be expected with a material of variable quality like concrete, on the probability that, on the average, the longer column will be more likely to have a weaker section than the shorter one. Although this is theoretically correct, a careful study of test results does not confirm any decided trend of results distinctly attributable to this cause. This can probably be explained by the greater strength of the lower part of long concrete columns, due to higher pressure of the wet concrete, when placed in one continuous operation, failure being almost wholly confined to the upper half of the column. There are few indications of decided difference in strength for lengths of less than 10 or 12 diameters, excluding the cube and cylinder, and possibly the trend of results shown in Fig. 2 would be better represented by a curve than the straight line. It is not apparent that the reduction factors for length for the various types of concrete columns are based on sufficient experimental evidence to justify any distinction between them. It is noted, particularly, that the vertically reinforced column, both with and without spiral, has been given the higher reduction factors, although its ratio of $\frac{L}{R}$ for a given length and outside diameter, would be smaller than that for the plain column or the column reinforced only with spirals. There is nothing in the data presented to indicate the necessity for change in the present usage of allowing a uniform stress up to lengths of 10 or 12 $\frac{L}{D}$ and a reduction beyond this point somewhat sharper than that indicated by the author's equations.

Cylinder Strength vs. Column Strength.—In considering the relation between cylinder strength and column strength, it is necessary to take into account the difference in moulding and curing conditions. Test columns are usually stored in air, with occasional sprinkling, during the first part of the seasoning period. Cylinders are stored in water, damp sand, or air for the whole or a part of the period. This greatly affects the strength, as does also the method of moulding, cylinders being subject to variation from difference in the greater amount of puddling and tamping generally given them than the larger columns. A statement of relation between cylinder strength and column strength obtaining in one laboratory, therefore, cannot be interpreted too broadly.

Longitudinally Reinforced Columns.—The relation just discussed has a bearing on the author's method of determining the ultimate stress in the rods of longitudinally reinforced columns by assuming equivalence of cylinder concrete strength and column concrete strength, and assigning the remainder

of the ultimate column load to the rod reinforcement. The calculated ultimate compressive stresses in mild steel rod reinforcement, as given in Column 7 of Table 5, range from 9 550 to 105 000 lb. per sq. in. For the rods in spirally hooped columns (Table 7, Column 9), a similar range is noted. This in itself indicates the method to be inapplicable with the small reinforcement ratios, as any considerable difference between assumption and fact as to the strength of the concrete unduly affects the computed steel stresses, and will make the average unreliable. As strains equivalent to the yield-point strain of steel of mild or structural grade are usually observed in tests of rod reinforced columns before failure, it would be better to accept this fact directly than to arrive at the same conclusion by using an average of erratic values, increased by an insecurely established length-strength relation.

Laterally Reinforced Concrete Columns.—The author prefaces the paper with the statement that the ultimate strength of laterally reinforced concrete is independent of the yield-point strength or the ultimate strength of the spiral reinforcement, although in the concluding statement, on page 1140, this is limited to the ultimate strength of the wire. Drawn steel of the carbon content used for spiral reinforcement has no decided yield point, the increase of rate of strain being gradual over a wide range of stress. Fig. 10 does not correctly represent a typical stress-strain graph for such steel. The criterion that limits the ultimate useful point of such lateral reinforcement is a certain unit deformation below the strain at maximum load obtaining in tension tests of the reinforcement. Measured strains in the lateral reinforcement near maximum load correspond to stresses, at times, as high as 80 or 90% of the ultimate strength of drawn steel, which must be considered to be above any yield point that can be assigned for the steel. Mild and medium rolled steels have definite yield points with elongations so large as to prejudice further aid to column strength.

For the strength of laterally reinforced concrete columns, the author assumes a form for his empirical Equation (21), of,

$$f'_{cs} = (1 + k\, r_s)\, f'_c$$

in which f'_c is the ultimate strength of the plain concrete assumed as equal to the cylinder strength; k, a constant; and, r_s, the percentage of lateral steel. This makes the increase in strength due to the reinforcement a linear function of the percentage of the spiral. From Equation (42) the unit load sustained is,

$$p = (e_r\, E_c)_m + X + \frac{r_s}{m}\, p_s,$$

in which $(e_r\, E_c)_m$ is the ultimate strength of the plain concrete; $r_s\, p_s$, the percentage and stress in the lateral reinforcement; m, a constant; and X represents the additional increase in strength due to the lateral reinforcement. It is seen that X is not a direct function of the amount of reinforcement. In the loading of a spirally reinforced column to failure, three stages can be distinguished, corresponding approximately with the three terms of Equation (42). Up to the load corresponding to the failure of the plain concrete, the stress-strain curve simulates nearly that of an unreinforced concrete column,

with low strains in the lateral reinforcement. Beyond this point, there is a transition stage, varying greatly in extent between different tests in which the rate of longitudinal and lateral deformation increases. This is the increment represented by the X term in Equation (42). The total load increment in this stage, although having some relation to the amount of lateral reinforcement present, is not directly proportional thereto, the smaller percentages up to 1% being proportionally more effective than higher percentages. In the third stage, the stress-strain lines maintain more nearly constant slopes up to points approaching failure, and the load increment sustained is nearly proportional to the quantity of lateral reinforcement present and the amount it can be stressed without incurring unduly high strains. It is in this region that the characteristics of a laterally restrained material are developed.

The author experiences difficulty in reconciling his empirical formula for laterally reinforced columns with the results of tests. He takes particular exception to the series of tests conducted at Lehigh University in 1911, as reported by Messrs. McKibben and Merrill and impugns their validity as having been made "for the purpose of proving and demonstrating the high strength to be obtained with spiral reinforcement."

The discrepancy between the results from the author's empirical formula (Equation (21)), and the experimental results is apparently in large part due to the assumption (1) of equivalence of plain column strength and cylinder strength, and (2) that the increment of strength of the laterally reinforced column is a direct function of the amount of reinforcement present, the validity of both of which has been discussed previously. The writer does not agree with Messrs. McKibben and Merrill in rejecting very high or very low test results as abnormal with a material as variable as concrete, as the range is as significant as the average values when properly interpreted. The net result of the eliminations made in their paper, which includes both low column values and high cylinder strengths, however, does not affect the merits of the spirally reinforced column by their method of presentation. It might also be noted that Mr. Tucker apparently accepts without objection the results of eight tests of spirally reinforced concrete in the same series, the aspersions being directed against the six, the results of which do not fall near the line in his Fig. 5. In obtaining his comparisons, he uses all the cylinder values, including also those rejected by Messrs. McKibben and Merrill as being unduly high, but even so, the actual column strengths for the six in the two refractory groups, B and H, having percentages of spiral of 0.46 and 0.96, respectively, range from 26 to 47% above the "theoretical". The points at issue are summarized in Table 31.

The author's formula from which he derives the "theoretical" strength on which the ratios in the last column of Table 6 are based, is, as determined from the line in Fig. 5,

$$f'_{ot} = (1 + 57\, r_s)\, f'_c,$$

in which f'_c is the cylinder strength. No reduction factor for length is applied. The average cylinder strength for the five groups considered is 2 433 lb. per sq. in., a normal average for laboratory-made 1:2:4 concrete at 70 to

80 days, and the average cylinder strength for the whole investigation is 2 427 lb. per sq. in. There is no indication in this case of any unduly low cylinder strength obtaining with the spirally reinforced columns, that would attribute an excessive strength increment for the reinforcement, nor have the results from the cylinders been utilized by any one connected with tests in any manner except as an approximate check on the quality of the concrete, until the publication of Mr. Tucker's paper, twelve years after the series was completed. Finally, the writer having been concerned with the planning and conduct of those tests is in position to state, if that is necessary, that they had no object other than that of investigation and that no conceivable condition existed to pervert this object.

TABLE 31.

Lehigh series.	Column numbers in Table 6 of paper.	$\frac{L}{D}$	Percentage of spiral reinforcement.	Average column strength, in pounds per square inch.	Average cylinder strength, in pounds per square inch.	Ratio: Cylinder to column strength.	Ratio: Column strength to strength computed by Equation (31).
B	(1) (2) (3)	8.6	0.47	3 339	1 965	1.70	1.38
H	(14) (15) (16)	4.3	0.97	4 823	2 374	2.03	1.31
C	(17) (18) (19)	8.7	0.97	4 750	3 103	1.53	0.99
J	(20) (21) (22)	17.2	0.97	3 602	2 165	1.66	1.07
D	(34) (35)	8.7	1.90	5 321	2 559	2.08	0.99
Average cylinder strength in above five groups.....			....	.....	2 433	....	....
Average cylinder strength for the whole investigation.....			....	.....	2 427	....	....

Theoretical Considerations.—The theoretical treatment of the action of concrete restrained by lateral reinforcement given by Equations (34) to (39), assumes a homogeneous, isotropic material of constant elastic properties. This does not hold for plain concrete and even less for laterally reinforced concrete throughout the loading range, as evidenced by the shape of the stress-strain lines. The assumptions imply constant values of modulus of elasticity, Poisson's ratio equal in all directions of strain, and a material able to resist up to the maximum all direct and induced compression, tension, and shear. For the given material, the treatment is of value only as outlining the main lines of action. The equations cannot be used for determining any constants from experimental data. For this purpose, it is necessary to include in the treatment assumptions more nearly coinciding with the true condition of the material; also, only the increments of load and deformation can be considered, over which the shape of the stress-strain curves corresponds with the assumptions. Where a linear variation of the one increment with the other is assumed, the available range is limited to the part of the loading where the stress-strain graphs approximate straight lines, which, for laterally reinforced columns, on the first loading will be within the third or final stage before the maximum is reached or closely approached.

The author obtains disconcerting values of Poisson's ratio apparently by taking directly the ratio of values of lateral and vertical deformation from the test curves. To have any meaning a constant must not change in value

within the pertaining range, even for considerable variation in reinforcement and resultant change in lateral pressures. It is apparent that the latter will affect the ratio between lateral and longitudinal deformations, even where the elastic constants do not change and make inapplicable as a constant of the material any ratios taken directly from the test curves for laterally reinforced columns. The values of these constants can be obtained only by the substitution of experimental stress and strain increments in equations correctly expressing their relations, considering the condition of the material at the given loading stage, and solving for their simultaneous values. By this method, the writer has obtained average and individual values for the modulus of elasticity and Poisson's ratio, as shown in Table 32, which covers a representative range in test data where both lateral and longitudinal deformations are reported.

TABLE 32.

Source.	Columns.	Percentage of lateral reinforcement.	Modulus of elasticity.	Poisson's ratio.
Am. Concrete Inst., 1915	(1), (1a), (1b)	0.98	368 000	0.31
"	(5)	1.88	307 000	0.43
Bulletin No. 46, Univ. of Wisconsin	L I	1.10	860 000	0.42
Lehigh tests, Am. Concrete Inst., 1916	Average Group B (3 tests)	0.47	353 000	0.28
"	Average Group C (3 tests)	0.97	330 000	0.29
"	Average Group D (3 tests)	1.90	257 000	0.36
"	Average Group E (3 tests)	3.80	312 000	0.42
Average		....	343 000	0.34

The author's objection to measurements of lateral deformation taken on 1 diameter or 2 diameters at right angles at the same point, as being over an infinitesimally small vertical gauge length, cannot be taken to hold fully, as the strains in the concrete tend to equalize the lateral detrusion, and it would be impossible for the section at the instruments to have deformation differing greatly from that of the neighboring concrete above and below, and an effective vertical gauge length of considerable extent can be taken to be present. The most accurate method, theoretically, involves a number of corresponding vertical and lateral gauge lengths, although the accuracy obtainable for individual readings would not be as great, due to necessary difference in types of available instruments that would have to be used.

Probability and Working Stress.—The author's use of present test data for reinforced concrete for establishing reliability as a final factor in determining working stress, although interesting, is open to the same objection as his use of test data for other purposes, namely, that present data cannot be taken as representative in range of the product as produced by the present methods. The same applies to the six-tenth relation between strength of field-placed and laboratory-made concrete obtained from a limited series of tests made about twenty years ago. It is apparent that both factors relate to the

same phase of the problem and should be represented by the same symbol of condition. It is also questionable whether the large reduction (0.50) due to repetitive loading is justified for the general purposes for which concrete is used. Investigators,* other than the one quoted in the paper, have found less reduction in ultimate strength from long-time application and repeated loads. The method outlined can be used as a basis for determining working stress only when supported by a sufficient amount of applicable test data.

H. Alden Foster,† Assoc. M. Am. Soc. C. E. (by letter).—The writer has taken much interest in Mr. Tucker's paper, but will confine his discussion to the general features, as he does not wish to take part in the discussion of the design of concrete columns.

Factors of Safety.—Mr. Tucker has given an excellent analysis of the principles which should govern the selection of working stresses, and the determination of the proper factor of safety. The factor of safety, as used in practical engineering, is composed of two parts: (1) a "factor of reliability" (to use the author's term), which is the ratio of the proper working stress to the average ultimate stress of the material; and (2) a "factor of ignorance", as it has been called, which covers all the other unknown features that influence the design of the structure.

Mr. Tucker claims that the factor of safety should not include this "factor of ignorance", that questions of overloading, distribution of loads throughout the structure, etc., should be treated as matters of design. It is possible that this could be done, at least to some extent; the designer may be able to determine what maximum loads will be carried by the structure. If he can say that a definite load will never be exceeded, he thereby reduces the "factor of ignorance" proportionally. This, however, cannot always be done; many structures, especially highway bridges, are standing, in spite of the heavy overloads that are carried, merely because the "factor of safety" in the design actually included an extra allowance for that very overload.

It is true that if the reliability of the material can be accurately determined, that part of the factor of safety could be correspondingly reduced. The specification writer, however, having practical conditions in mind, would still have to consider the unknown features affecting the design before he could select the proper factor of safety. The writer believes, however, that a scientific study of engineering materials, along the lines suggested by Mr. Tucker, would result in the determination of factors of safety which would have a logical basis for existence.

Possibly the best method would be to select a factor of safety based on experiments and the reliability theory, but to provide for increasing this factor of safety in a definite manner according to the type of structure or the manner of its use.

Reliability.—The author has given a very clear explanation of the theory of probability as applied to physical measurements, basing his analysis on the

* See Messrs. Hatt, Am. Soc. for Testing Materials, 1907, and Berry, Am. Soc. for Testing Materials, 1909–10.

† Structural Designer, D. P. Robinson & Co., New York City.

normal or "symmetrical" law of error. As explained in the writer's paper on "Theoretical Frequency Curves and Their Application to Engineering Problems"* certain classes of physical measurements do not follow the normal law of error. It is necessary in this case to distinguish between measurements of the same quality, and those made on similar qualities of different objects. For example, if a series of measurements are made to determine the length of a base line, the true quality measured is the same for each measurement, namely, the true length of the base line. All the actual measurements will differ from the true value, but the errors are compensating and, as has been learned by experience, they follow the normal law of error.

When measurements are made on similar qualities of different objects, the conditions are rather different. Assuming that each measurement is correct in itself, the different measurements, nevertheless, will show considerable variations. Thus, if several test specimens are taken from the same heat of steel, the resulting values for ultimate strength determined for the several specimens will all differ, regardless of how accurate the testing machine may be. Obviously, the measurements are not made on the same quality, but merely on similar qualities. Experience has shown that, as a rule, the resulting values will differ from the mean value not according to the normal law of error, but according to the so-called "unsymmetrical probability law".

As Mr. Tucker has pointed out, observations of this type are not symmetrically located with respect to their mean, because there is a definite lower limit beyond which they cannot extend, namely, zero, whereas there is no such definite upper limit. Under certain conditions, the lower limit may lie between zero and the mean, for example, when the condition of the material is largely controlled by inspection. It can, however, never be less than zero.

It is shown in the writer's paper referred to that any set of engineering data which tends to follow the unsymmetrical probability law can be classified by determining a "coefficient of variation" and a "coefficient of skew", as follows:

If v = the variation of any observed value from the mean of all the observed values, expressed in terms of the mean; and

n = number of observations,

the coefficient of variation equals

$$c\,v = \sqrt{\frac{\Sigma\, v^2}{n-1}},$$

the coefficient of skew equals

$$c\,s = \frac{\Sigma\, v^3}{(n-1)\,(c\,v)^3},$$

and the reliability number, used by Mr. Tucker,

$$N_r = 1.00 - 1.415\,t \times c\,v.$$

With these coefficients, a curve showing the distribution of the observations with respect to the mean can be readily constructed. The resulting curve, known as a "duration curve" or "curve of total probability", shows the number of times or percentage-of-time in which the observed value equalled or exceeded

* *Proceedings*, Am. Soc. C. E., May, 1923, Papers and Discussions, p. 825.

any given amount. Thus, from this curve could be found the value of the ultimate strength of steel which was exceeded in 99 out of 100 specimens, or in 999 out of 1 000 specimens. The latter value would be at 99.9% of the time.

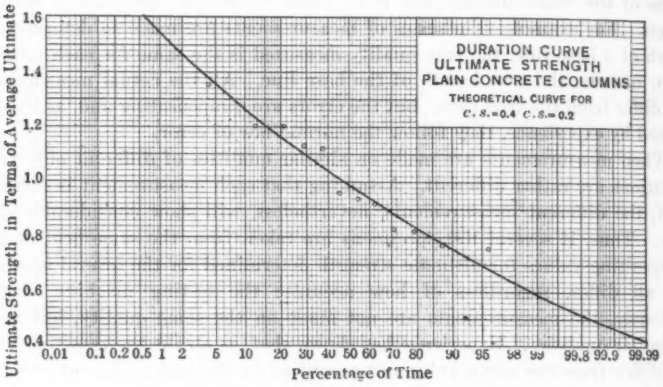


FIG. 19.

To illustrate, the writer has taken the data given in Table 1, for plain concrete columns, using the ultimate stress of twelve specimens, given in pounds per square inch. The values are first arranged in order of magnitude, and then expressed in terms of their mean. The variation from the mean, v,

TABLE 33.—PLAIN CONCRETE COLUMNS

Ultimate stress, in pounds per square inch.	In order of magnitude.	In terms of mean.	v.	v^2.	v^3.
2 045	2 990	1.35	+ 0.35	0.1225	+ 0.0428
2 137	2 660	1.20	0.20	0.0400	0.0080
2 087	2 660	1.20	0.20	0.0400	0.0080
1 698	2 500	1.13	0.13	0.0169	0.0022
1 836	2 480	1.12	+ 0.12	0.0144	+ 0.0017
1 676	2 137	0.96	− 0.04	0.0016	− 0.0001
1 819	2 087	0.94	0.06	0.0036	0.0002
2 660	2 045	0.92	0.08	0.0064	0.0005
2 660	1 836	0.83	0.17	0.0289	0.0049
2 480	1 819	0.82	0.18	0.0296	0.0053
2 990	1 698	0.77	0.23	0.0529	0.0122
2 500	1 676	0.76	− 0.24	0.0576	− 0.0138
Mean = 2 215	.....	....	....	$\Sigma v^2 = 0.4144$	+ 0.0627 − 0.0370 $\Sigma v^3 = +0.0257$

$$c\,v = \sqrt{\frac{0.4144}{11}} = 0.19$$

$$c\,s = \frac{0.0257}{11 \times 0.00716} = 0.33$$

also v^2 and v^3, is given. From these values, Σv^2 and Σv^3 are computed; and the $c\,v$ and $c\,s$ for these data are readily computed. The data are plotted on "probability paper", in Fig. 19 and the theoretical duration curve for a $c\,v$ of 0.2 and a $c\,s$ of 0.4 is obtained and plotted, by the method explained in the writer's paper previously mentioned.

The data selected for this example are not very complete, and they also were obtained for columns of various lengths and different materials. Consequently, the results obtained cannot be considered of any importance, quantitatively. Fig. 19 is given merely to illustrate the method of studying such data and to show that the observed values tend to follow the unsymmetrical law. The chief purpose of this method is to determine the value of the column strength which will probably be exceeded in every case. A value may be selected which will occur with a probability of only $\frac{1}{100\,000}$ or $\frac{1}{10\,000\,000}$. The latter would be exceeded during 99.99999% of the time.

In the analysis of unsymmetrical frequency curves, the writer did not extend his calculations for the constants of the curve beyond 99.9% of the time. He will arrange to compute an extension of Table 2 of his paper previously mentioned, so that values for these very small probabilities may be readily obtained.

In the example given, the data are seen to have a small coefficient of skew. This would very likely be true of most test series, showing that they do not differ so very much from the normal law of error. This slight skewness, however, is important when very small probabilities are considered. Fig. 4 of the writer's paper shows that, as the percentage-of-time is increased, the divergence of the skew curves from the normal curve increases. With zero skewness, if the percentage-of-time is increased sufficiently, the value of the observation will become less than zero, which, of course, is an impossible condition. The writer has shown in his paper that the coefficient of skew must always be greater than twice the coefficient of variation. This condition can be used as a check on the values for $c\,v$ and $c\,s$ computed from the data. In Fig. 19, a $c\,s$ of 0.4 was used to obtain the theoretical curve as the $c\,v$ was 0.2, although the $c\,s$ as computed from the data was only equal to 0.33.

The writer believes that by examining the data along the lines suggested, more satisfactory results for the reliability number can be obtained. Merely to compute $\sqrt{\frac{\Sigma v^2}{n-1}}$ and substitute in Mr. Tucker's formula for reliability does not indicate whether or not the data are reasonable. A graphical plotting of the duration curve, from the actual data, and a comparison with the theoretical curve, shows quite definitely whether the observations correspond to the theory. By extending the theoretical curve to high percentages of time, a definite value of the reliability can be obtained, which agrees with the theory and is based on the observed data. It is also possible to select a value for the reliability for any desired percentage-of-time or frequency of occurrence.

Having selected the reliability numbers for a given grade of steel and a concrete of a certain aggregate and mix, it remains to determine the reliability of the column in which the steel and concrete are used. Suppose that the reliability of the steel is N_s and that of the concrete is N_c. The reliability of the column will depend not only on N_s and N_c, but also on the extent to which the strength of the column is influenced by the steel and the concrete. Assume, for example, that theoretical or practical considerations show that the average ultimate strength of the column is equal to

$$k_1 S + k_2 C$$

in which,

S = average ultimate strength of the steel; and
C = average ultimate strength of the concrete;

and k_1 and k_2 are factors determined by the percentage of steel, dimensions of the column, etc. Then, in the extreme case in which the steel used has a strength of $N_s S$ and the concrete a strength of $N_c C$, the column as a whole will have an ultimate strength of

$$k_1 S N_s + k_2 C N_c$$

and the reliability of the column is equal to

$$\frac{k_1 S N_s + k_2 C N_c}{k_1 S + k_2 C}$$

It is impossible, therefore, to determine the reliability of the column from N_s and N_c unless k_1 and k_2 and the average strength of the steel and concrete are also known. Probably the most practical method to obtain the reliability of a compound material, like a reinforced concrete column, would be to compute it directly from tests of that type of column, rather than by trying to use the reliability numbers of the steel and concrete as determined separately.

JOHN TUCKER, JR.,* ESQ. (by letter).—It is impossible to determine the strength of structural elements, especially of reinforced concrete, by a purely abstract analysis without direct experimental basis of tests on similar elements, owing to the influence of its several physical constants and to the great variation of these constants due to the effect of the non-homogeneity of concrete on the strength of the element. If any pretense at scientific method is made, the design of the structure should be based directly on these tests and in no uncertain or equivocal manner.† It is difficult fully to impress the research or laboratory worker with the importance of the knowledge of the effect of the various imperfections occurring in practice (some avoidable, many more unavoidable) on the strength of the structural element, and the impossibility of eliminating many of the imperfections. Too great emphasis cannot be placed on the importance of searching for and of obtaining this knowledge. It is also difficult to impress the field engineer, of necessity working with the relatively crude field technique, of the importance and possible great value of laboratory tests and research.

* Pittsburgh, Pa.

† See Section IV of the paper, p. 1126.

The process of the determination of the safe proportions and composition for a structural element is divided into the two parts:

1.—The determination of the maximum load that a material or structural element will support.

2.—The design of the structure so that, at a maximum, the stress or load imposed on the material or structural element will not exceed the load or stress as determined by Part 1.

The information necessary for the first part is supplied by specimens prepared in the laboratory, and built with the care and precision impossible or impractical to achieve in the final structure. Still another factor is necessary if the design is based on carefully prepared laboratory specimens, that is, that between the strength of the laboratory prepared and the practically prepared element. The only variable that is different in the practically prepared column is the strength of the concrete. The strength of the reinforcement even for field structures can be determined and controlled very accurately. The ultimate strength of the various types of reinforced concrete columns is a function of only one property of the concrete in them, that is, the strength of the concrete as exhibited in test cylinders. Therefore, if the strength and strength reliability of the concrete of the column in the finished structure are known, the probable strength of the column in the finished structure and the minimum strength which will, with certainty, be developed, can be determined by the equations evolved from the strengths of specimens made in the laboratory and fabricated with special care.

Cylinder test specimens of the concrete, as used in the finished structure, form a sufficient basis for obtaining satisfactory knowledge of the strength of the column in that structure, and eliminate the effect of the difference in strength of a certain aggregate as made commercially and as made in the laboratory. When check test cylinders are not made, which is a procedure not recommended by the writer, but which is frequently the only practical one, then the relation between the strength of laboratory prepared and commercially prepared concrete should be known. The information on this question, published in 1915,* and not "some twenty years ago", as Mr. Ingberg stated in his discussion, is not to be considered as final. The series is admittedly small, but it is necessary to have some indication of the relative strengths, and this series gives the best available indication. The omission of check test cylinders was recognized in the paper as a "procedure not to be recommended". It is important once more to emphasize this statement.

The second essential in the process of the determination of the safe proportions and composition of a structural element, that of proper design so that the maximum stresses or loads to be impressed on the individual structural element will be equal to the safe stress or load on it, and the fundamental determination of the actual stress produced in it by all possible loading of the structure, is a consideration independent of the strength developed by perfect structural elements prepared and tested in the laboratory. Pertinent tests can be made in the laboratory to establish the essentials necessary for the

* *Technologic Paper No. 58*, U. S. Bureau of Standards, 1915.

design, as, for example, the necessity of ensuring the proper transfer of stress into the rods of a rod reinforced column, which is accomplished by prolongation of the rods into the column pedestal, etc. This is an important consideration in the design of longitudinally reinforced concrete columns, which Mr. O'Rourke emphasized in his discussion. Investigation of one phase of the actual stresses in the structure produced by the loading is being made in a thorough manner by the Special Committee to Report on Stresses in Railroad Track. The actual stresses have been, and are being, measured under trains in motion for various conditions of road-bed on straight and curved track. In its Second Progress Report, the Committee has discussed the "variation in measured stress and the relation between average and maximum stress", and, in the formula for the stress in the rail, a "variability factor" is included.* The probable variation corresponding to the standard "probable error" was found to be approximately 18 per cent. The value corresponding to an infinitely small probability was not attempted. The method for determining the reliability number, or minimum possible strength of Section III of the paper, may be applied in the determination of the maximum possible stress correlated to an infinitely small possibility, giving an approximate value of 125% of the mean measured stress in the rail. The curve (Fig. 141) representative of the points showing the variation of the measured stress, given in the report, indicates a skew frequency or probability curve. As the determination of the form of the curve can be made with considerable accuracy because of the large number of points which have been, and can be obtained, calculations of the maximum stress to be expected should preferably be made by the method indicated by Mr. Foster in his paper, "Theoretical Frequency Curves and Their Application to Engineering Problems",† and by Mr. Cox in his discussion of the writer's paper.

The writer's paper deals only with the first part of the problem of the determination of the safe proportions and composition for a structural element, namely, the determination of the maximum load that a material or structural element will support safely. The second part of the problem which was omitted, requires far more experimental data which are much more difficult to obtain than that required for the first part of the problem. The lack of definite knowledge of this second part does not make the determination of the proper proportioning of the structure unsafe. It merely prevents the utilization of the minimum of material for absolute safety, as, for example, the design, so that a predesignated eccentricity of column loading could not be exceeded. Experimental observation would show the maximum value of the variable to be expected, which would be less than the value which would have to be assumed in design without that experimental data.

Of necessity, the equations derived in Section I of the paper are not rigidly exact, as they are based on a limited number of test columns. They give, however, the best indication of the strength of a concrete column, based on the strength of the same concrete in test cylinders, that can be obtained

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), Section 54, pp. 1378–1382.
† *Proceedings*, Am. Soc. C. E., May, 1923, Papers and Discussions, p. 825.

from the available test columns, and also the best indication of the strength that can be obtained from any source. The equations developed in Section IV of the paper give the best possible indication of the safe load or stress for the various types of reinforced concrete columns based on the strength of the concrete in tests cylinders, that can be obtained from the available data, and also the best available indication that can be obtained of the safe loading from any source. The loading or stresses indicated by the equations of Section IV are safe, because the applications of the various factors entering into the determination were always made on the side of safety. For example, the equation giving the variation in strength was the Gaussian, not the skew form of Karl Pearson, the more probable form to be expected and so stated on page 1121 of the paper.

Mr. Ingberg contends that "the method outlined can be used as a basis for determining working stresses only when supported by a sufficient number of applicable test data", and Mr. Godfrey objects to the value of the safe stresses as solely applicable to the predicted strength of laboratory specimens. If the writer's equations do not give the best values as determined by future investigation, they give the best values that can be determined from the information available. In previous derivations of values for safe stresses, as set forth in detail on pages 1127 to 1131, inclusive, the same laboratory strengths of columns are used as a basis, and the effect of all operating factors accepted by them as unknowables are included in a conglomerate factor of safety. The strength of laboratory test columns must be the basis of the design strength in any rational plan. The difference in the plan developed in the paper and the previous solutions, is the method of treating the unknowable quantities. Surely Mr. Ingberg does not prefer, for the factor of safety, a guess in preference to that mathematically determined by experimental data, however limited that data may be?

The group of tests* by Professor Berry of nine columns "made under the actual conditions of building construction" afford an excellent comparison of the actual strength developed and the strength indicated by the equations of Section I of the writer's paper. There were only three comparison test cylinders, a very small number on which to base column strength. By comparison with the plain concrete columns tested, it is probable that the average strength of the cylinders is a trifle low. Correcting the average ultimate cylinder strength to the age of the individual columns, and computing the probable strength by means of the equations in Section IV, the results given in Table 34 are obtained.

The strength of the longitudinally reinforced column is given by the equation:

$$\frac{P}{A} = \left[1.28 - 0.034\ \frac{L}{D}\right][r_c\, f'_c + 31\,000\, r_r],$$

which is Equation (55) when the product, $N_r\, L_e = 1$.

* *Engineering Record*, February 19, 1910, p. 206.

The strength of the spiral and rod reinforced column is given by the equation:

$$\frac{P}{A} = \left[1.20 - 0.022\,\frac{L}{D}\right][(1 - r_r)\,(1 + 0.573\,r_s)\,f'_c + 36\,500\,r_r],$$

which is Equation (59) when the product, $N_r\,L_e = 1$.

TABLE 34.

Type of Column.*	Ratio of Actual to Calculated Strength.	
	Individual column.	Average for like columns.
Plain concrete	1.15 0.99 1.15	1.10
Reinforced with 1.15% of longitudinal rods	1.115 1.000 1.010	1.032
Reinforced with 1.99% longitudinal rods and 1.71% of spiral, wound on a 12 in. diameter	1.02 1.00 1.015	1.012

* The columns were 14 in. square and 12 ft. long.

The Function of the Test Cylinder.—To be of service in design, the equation expressing the ultimate compressive strength must be a function of readily measured variables which express the strength of the column components. The possible variables expressing the strength of the concrete in the column are:

1.—Full-size unreinforced concrete of the same fabrication and of the same dimensions as the reinforced column.

2.—*The proportions of the components of the concrete, and their physical properties.

3.—Test specimens of smaller dimension, either from the same proportions, laboratory made, or preferably from the identical batches from which the finished structure is made: (*a*), Cubes; and (*b*), cylinders.

It is manifestly absurd to test full-size specimens for every structure. The proportions of the component materials, etc., give only a crude indication, and the concrete strength is dependent on so many variables, especially the quantity of water and the thoroughness of mixing, that at best it is a very crude method. The test cube has only one apparent advantage in that the variation in strength is lower than that in cylinders, or even in full-size test columns, but this invariance in strength is exhibited when it is known that the actual strength of the concrete differs, as can be appreciated by analysis of a large group of tests by Professor Rudeloff.† Hence, the strength of the cube is only a poor indication of the actual strength of the

* An option in the 1921 Tentative Specifications of the Joint Committee on Concrete and Reinforced Concrete, *Proceedings*, Am. Soc. C. E., August, 1921, p. 59.

† *Deutscher Ausschuss für Eisenbeton*, Heft 25, 1913.

concrete. As the relation of the strength of the cube to that of the concrete is not a constant, and as the strength exhibited by the cube per unit area is much greater than the actual strength capable of being developed unassisted by the concrete in the column (see page 1078), it is a poor type of test specimen for use as a basis for determining column strength.

As the equations developed in Section I of the paper were intended for utilization as the basis of the derivation of the safe stresses in Section IV, they were based on, or given in, terms of the ultimate strength of concrete of the same batch as the columns, in the form of test cylinders, as the selected variable. The data on numerous test columns from which other information could be obtained or by which theorems developed in the paper could be further substantiated, were, for brevity, omitted from the paper where the tests included no comparison test cylinders, and, therefore, could not be included in the derivation of the strength equations. Where applicable, a number of these tests will be given in the appropriate part of this discussion.

Having developed the equations of Section IV expressing the safe column stresses or loads in terms of the ultimate strength of test cylinders made from the same batch, test cylinders made of the concrete actually used in the structure give an incontestable value showing the actual strength of the concrete in the structure. Mr. Martin questions the value of field cylinder tests because "it is the usual practice to cast two or three pairs of cylinders from a day's run consisting of 200 to 400 batches." For comparison he continues, "in testing open-hearth steel at the mill, the engineer requires tests to be made of each charge or heat, and would not regard very highly a report of tests made from one heat out of a hundred, although that is all a field cylinder test of concrete is."

Granting that the facts as stated by Mr. Martin are correct: Should abuses be tolerated because they are customary? It is true that field cylinder tests may be "an indication only of the quality [strength] of the concrete that can be made from a particular combination of aggregates and cement", as Mr. Martin contends, but can they not be used with more purpose as the exact indications of the strength of the concrete that is made from the particular combination of aggregates and cement, and full advantage taken of this complete knowledge of the strength? If the field test specimens are insufficient to demonstrate exactly the strength of the concrete in the finished structure, a reduction in permissible stress on the concrete element must be made.

No minimum number of field test cylinders was suggested in the paper. The number of specimens required to determine definitely the strength of the concrete in the finished structure, and the variation in that strength, is sufficient. If there is accurate control of proportioning, especially of the water content, and of the time for mixing, a relatively small number of test cylinders would be adequate, say, one specimen for every six batches, to be taken at a random mix to insure that the operator, knowing that test cylinders would be made from a certain mix, would not prepare a mix better than the average run.

Certain factors make the necessary number of test specimens of concrete and of open-hearth steel very different. The physical properties of steel other than its strength are usually of even more importance, whereas the strength of concrete is the only important physical property. Steel, after making, undergoes other fabrication in forging, rolling, drawing, and its composition must be closely limited in order to permit perfect fabrication and to give a finished product of the desired quality. Factors other than the desirability of the number of test specimens are also necessary in order to determine with accuracy the strength, and variation in strength, of a product. These factors are custom, the relative ease of making tests, the economic factor, governed chiefly by the relative cost of test specimens and finished product, and the relative economy in saving of material effected by the use of test specimens.

Variation of the Strength of Concrete Columns with Variation in Length.—Objection has been made in the discussion of the paper to the equations giving the reduction in strength of the concrete column (both plain and reinforced) with increase in strength. The true relation between length and strength of the concrete column is obscured by the large natural variation in the strength of the concrete. Similarly, as an individual concrete column, reinforced with rods or spiral, may show less strength than an individual unreinforced concrete column made from the same batch of concrete, so an individual long column may show less strength than an individual short column. The steel column (Fig. 4), and the column of the von Emperger type (Fig. 6), show clearly a decrease in strength with length, the decrease being apparent even for the shortest lengths. Is it not logical to assume that the same phenomenon, although not necessarily to the same degree, manifests itself in the concrete column?

Euler's column formula, $P = \frac{\pi^2 E I}{l^2}$, derived for a "thin rod, *i. e.*, a mathematical line endowed with flexural rigidity",* is hardly applicable to concrete columns. Euler's formula, if accepted as correct, has application for a minimum length of 20 diameters for a steel column,† a length greater than which concrete columns are seldom built. Professor Deimel‡ has shown the Euler formula to be seriously in error for long columns of practical dimensions, and has given an excellent derivation of the correct long-column formula, which has application for a minimum length much longer than that indicated by the Euler formula. The Euler and Deimel formulas indicate that, for columns shorter than the minimum length for which the formula is applicable, the column strength is constant, and equal to the yield-point stress of the material per unit column area. The minimum length of the wrought-iron column by the Deimel formula is $170 \frac{l}{r}$, a comparatively long column.

* "The Theory of Columns," by R. F. Deimel, *Stevens Indicator*, January, 1912, Vol. XXIX, No. 1.

† "Theory of Structures", by A. Morley, 1912 Edition, p. 279.

‡ *Stevens Indicator*, January, 1912, Vol. XXIX, No. 1.

Rankine's semi-empirical formula,* a combination based on the Euler formula for the limiting condition of the very long column, and a strength equal to the product of the column area and the ultimate stress of the material as the limiting condition for the very short column, can have no more application to the concrete column of the usual dimensions than the Euler formula.

Of the group of specifications for the decrease in allowable column load with increase in length, collected by Professor Giesecke,† and presented by Mr. Feld, on page 1159, the German formula, as might be expected, is the Euler formula with the introduction of the factor of safety, $s = 10$. The French and Swiss formulas are adaptations of the Rankine formula, thus eliminating all three of these formulas as a basis of comparison with those for concrete columns. For comparison with the formula of the 1921 Tentative Specification of the Joint Committee on Concrete and Reinforced Concrete,

$$P = P' \left(1.33 - 0.033 \frac{L}{D}\right),$$

of the equations developed by the writer, in place of those given by Mr. Feld, the three following should be selected:

For plain concrete:

$$P = P' \left(1.17 - 0.020 \frac{L}{D}\right)$$

For rod-reinforced concrete:

$$P = P' \left(1.28 - 0.034 \frac{L}{D}\right)$$

For spiral and rod-reinforced concrete:

$$P = P' \left(1.20 - 0.022 \frac{L}{D}\right)$$

Moreover, the equations previously mentioned as given by Mr. Feld, except the last three, are intended solely for safe load determination, may not be intended to represent the true reduction in column strength with length, and cannot be used for comparison with other than the same type of column for which the equation had, if any, empirical basis.

Mr. Feld cites the 1897 tests made at the Watertown Arsenal, presenting the data (Table 14) and a note from the description of the tests, in order to indicate that there is probably no difference in strength with length. The tests were made in such a manner, however, as to obscure, if not entirely to invalidate, them as a means of determining the true reduction in strength with increase in length. The columns, which were from 2 to 14 ft. long and 12 in. square, were tested horizontally. The specimens, 6 to 14 ft. in length, were counterweighted with 460 to 760 lb. from a wire rope around the middle of the column. The results, therefore, do not give the true strength-length relation. Nevertheless, correcting the column strengths to the same age, the

* For derivation, see Morley's "Theory of Structures", 1912 Edition, p. 279.

† *Engineering News-Record*, Vol. 89 (1922), pp. 274–276.

points representing the average strengths of the columns of the same length give the representative line:

$$P = P' \left(1.00 - 0.0091 \frac{L}{D}\right),$$

which shows, even with the columns laterally restrained as stated, more than 50% of the reduction in strength given by the writer's Equation (4),

$$P = P' \left(1.000 - 0.0171 \frac{L}{D}\right).$$

If the difference in age is not corrected, the equation representing the group of test columns is:

$$P = P' \left(1.000 - 0.0152 \frac{L}{D}\right).$$

The other group of tests cited by Mr. Feld, and made by Professor Giesecke at the University of Texas,* can have but little weight. Quoting Professor Giesecke:

"* * * they also show that the results of the tests are not as uniform as they should be in a carefully conducted investigation. The writer believes that the irregularity of the results are due to a difference in the water content of the concrete due to the particular location of the column in the mould, and to the location of the mould."

The columns are small, especially in the short (3-in.) dimension, relative to the size of the large aggregate, that is, a 3 by 6-in. cross-section for a ¾-in. aggregate. Being moulded horizontally with the 3-in. side on top—another detrimental factor—resulted in the failure of the column about the 3-in. instead of, as was to be expected, the 6-in. axis. Concrete columns of unsymmetrical cross-section are the exception, and it is reasonable to expect that the reduction in strength with length for a section of the proportions of 1 : 2, differs from that of a section having a 1:1 proportion. An unfavorable factor of the tests, which resulted from the method of moulding horizontally, is the method of failure of the columns, which was around the 3-in. axis, instead of around the 6-in. axis. After testing a few columns, the loading was applied ½ in. off center to overcome this defect. Considering the method of failure, it is indeed questionable whether the length-diameter ratio can be computed as based on the long (6-in.) axis.

By averaging the strength of columns of the same length, on the basis of $\frac{L}{D}$ of the 6-in. axis, the points give the two representative lines for the two groups of columns in the University of Texas tests:

$$P = P' \left(1.000 - 0.0055 \frac{L}{D}\right)$$

$$P = P' \left(1.000 - 0.0070 \frac{L}{D}\right)$$

The group of columns tested by Professor Giesecke have great value in indicating the influence of a wet concrete on the strength of beams. For

* *Engineering News-Record*, Vol. 89, 1922, pp. 274–276.

economy, the outer "top" fiber of the beam should contain the strongest concrete, yet these tests show that the weakest concrete is at this point.

Mr. Feld states:

"Certainly there is no reason for allowing a less reduction for plain concrete columns than for the reinforced columns, as such reduction is meant to allow for the tension at the outermost fiber caused by flexure."

The reduction of strength given in the paper is the actual reduction determined from the group of test columns analyzed, and for no other reason. Moreover, columns within the category of used and usable concrete columns, plain and reinforced, do not fail by buckling to cause "tension in the extreme fiber". Failure by buckling can only occur beyond the minimum length for which Professor Deimel's column formula is applicable. It is well to emphasize here the correct interpretation of the buckling of test concrete columns. The buckling is a post ultimate phase of the test, and occurs after the column has sustained its maximum load. In the tests of three spirally reinforced columns in a group, Professor Berry points out* that one column, after receiving the maximum load, buckled 6 in. in a total column length of 12 ft., yet sustained more than 50% of the maximum load (2 500 lb. per sq. in. on core area).

Mr. Reeves contends that "the ratio of length to diameter has no place in any concrete column formula". He quotes Professor Talbot and Mr. Lord as follows: "* * * the stress taken by the concrete may be considered to be nearly independent of the slenderness ratio of the column, within the lengths tested * * *". This is not a general statement about concrete columns, but refers specifically to the structural steel column cast in spirally reinforced concrete. The constance of the strength added to the steel column by the surrounding concrete was recognized and expressed in the paper by the statement:

"* * * Professor Talbot draws the conclusion that the strength of the column is equal to that of the structural steel element plus the strength per unit area of concrete equal to the ultimate strength of the concrete as tested in 8 by 16 in. test cylinders."

The strength of the column as a whole, however, decreases, because the strength of the structural steel column decreases. The quotation as given by Mr. Reeves is, therefore, misleading.

In order to sustain his contention, Mr. Reeves also quotes Messrs. McKibben and Merrill, as follows:

"The average compressive strengths of 8 in. by 16-in. cylinders and 5-ft. columns, 14 in. in diameter, are very nearly equal, while the strength of 10-ft. and 20-ft. columns are approximately equal, amounting to 81% of the strength of cylinders."

For clarity, this statement may be summarized thus:

Length of column, in feet.	Column strength, in percentage of cylinder strength.
5	100 (very nearly)
10	81 (approximately)
20	81 "

* *Engineering Record*, February 19, 1910, pp. 206–208.

Although this tabulation does not clearly indicate the actual reduction in strength with length for the unreinforced concrete column, yet it cannot, by any perversion, be construed as proof that the strength is independent of length.

For further affirmation of his statement that the ratio of column length to diameter has no place in any concrete column formula, Mr. Reeves, on page 1171, quotes Professor Talbot, as follows:

"It seems hardly necessary to advance the idea here that for concrete columns used in ordinary building construction little attention need be given to the relation between length and lateral dimensions after a length of a few diameters has been reached. * * * Even for 15 diameters we may readily conclude from the calculated results of long column formulas and also from the small lateral deformation found in the columns tested that the difference in strength between a column 15 diameters long and one 5 diameters long is less than the variation among several columns of the same length. * * * For columns eccentrically loaded, the effect of any eccentricity is generally large in comparison with the lateral deflection used in the Euler analysis, and it may be said to be generally independent of the length of the column. Clearly, for conditions of ordinary design, a formula for plain columns or for columns reinforced with longitudinal rods need not include the ratio of length to lateral dimensions."

Professor Talbot does not state that the strength of concrete columns is constant within certain length limits, but that the difference is less than the possible difference in strength of columns of the same diameter due to the natural variation in strength of concrete, and that, therefore, "for conditions of ordinary design, a formula for plain columns or for columns reinforced with longitudinal rods need not include the ratio of length to lateral dimensions". This is solely a pronouncement of a policy of simplification and is independent of the actual variation in strength of the column with length. A preferable policy would include the effect of the length-diameter ratio in the formula for the safe strength of the column, delegating whatever simplification with concomitant loss in available strength for the shorter size column, to the designer, who, in the special case, may take advantage of any small additional strength, or disregard it for expediency.

The Euler formula has been shown to be in serious error even for "long" columns. Moreover, a "long" column formula (Professor Deimel's formula) does not apply to concrete columns of the dimensions ordinarily used, nor has there been any relationship established between the lateral deformation for a concrete column and its length and strength. The conclusions which Professor Talbot draws from inapplicable and erroneous formulas concerning the magnitude of the reduction in column strength, therefore, are clearly unwarranted, and can have no weight.

Unreinforced Concrete Columns.—The analysis devoted to unreinforced concrete columns in the paper, although short, was sufficient to develop fully the available data so that the safe working stresses for this type of column could be computed by the methods of Sections III and IV. It was thought that there would not be sufficient interest in the subject to justify a more extended treatment, and the curtailment should in no way be construed as an inference that the unreinforced concrete column is of no value or of small

value as a structural element. Because of the interest which has since been shown, the safe stresses for the unreinforced concrete column have been determined by means of the appropriate equations of the paper.

Substituting Equation (5), as the value of $F(V.Mf'_c)$ in Equation (54), we obtain:

$$\frac{P_m}{A} = 1.17\, N_r\, L_e\, f'_c \left(1.00 - 0.0171\, \frac{L}{D}\right) \ldots\ldots\ldots\ldots (89)$$

For unreinforced concrete columns:

$$N_r = 0.55^*$$
$$L_e = 0.50\dagger$$

Therefore,

$$\frac{P_m}{A} = 0.32\, f'_c \left(1.00 - 1.0171\, \frac{L}{D}\right) \ldots\ldots\ldots\ldots\ldots (90)$$

Equation (90) expresses, for values of $\frac{L}{D}$ not greater than 16.6, the safe stress for unreinforced concrete columns in terms of length, diameter, and ultimate strength of the concrete from the same batch as that in the column, as determined in the test cylinder. The empirical Equation (5) cannot be used for values of the variables beyond the range of the experimental variable,‡ hence the derived Equation (90) must have the same restriction with the upper limit, 16.6, for the length-diameter ratio.

If no check test cylinders are made, or if the test cylinders are carefully fabricated, laboratory specimens made from the same proportions of aggregate as the concrete in the column, as the commercially fabricated concrete in the column cannot be expected to be greater than $0.6\, f'_c$, the safe working stress is given by the equation:

$$\frac{P_m}{A} = 0.193\, f'_c \left(1.00 - 0.0171\, \frac{L}{D}\right) \ldots\ldots\ldots\ldots\ldots (91)$$

Table 35 gives safe stresses calculated by Equations (90) and (91).

TABLE 35.—SAFE WORKING STRESSES FOR UNREINFORCED CONCRETE COLUMNS, EXPRESSED IN PERCENTAGE OF TEST CYLINDER STRENGTH.

	By Equation (90).	By Equation (91).
6	28.9	17.3
8	27.8	16.7
10	26.7	16.0
12	25.5	15.3
14	24.8	14.9
16	23.3	14.0

NOTE.—The 1917 Tentative Specifications of the Joint Committee on Concrete and Reinforced Concrete give 22½% for "piers", $\frac{L}{D}$ a maximum of 6; and the 1921 Tentative Specifications give 25% for a "pedestal", $\frac{L}{D}$ a maximum of 3.

* See Table 10.

† See foot note, p. 1132.

‡ See foot note, p. 1075.

The columns to be analyzed for the determination of the effect of variation of strength on length must have some means of comparison. The strength of a column in a structure must have some definite and readily measured quantity as an index. The mix is only a crude indication of the strength, and tests of full-size columns for every structure would be absurd. The test cylinder is a means of comparison that can also be used as the index of the strength of the concrete in the column, and, therefore, was chosen as the basis for co-ordinating for analysis the available test columns.

Only those columns for which test cylinders were made from the same batch of concrete could be included in the analysis; it is manifestly impossible to include the others. For example, the twenty-seven unreinforced concrete columns tested at the Watertown Arsenal, from 1902 to 1908, inclusive, suggested by Mr. Buel, on page 1164, are all of the same length and diameter, have no test cylinders for comparison, and are made of various mixes and aggregates. They serve, therefore, solely to establish a rough relationship between strength and proportion of mix.

Of the group of unreinforced concrete columns given in *Bulletin No. 20,* University of Illinois, only nine columns have duplicate test cylinders. Although the columns were tested at the age of 61 to 72 days, with one exception at 201 days, the cylinders were tested at the age of 185 to 338 days. Owing to the difference in age and consequent discrepancy in strength, the cylinder strengths must be corrected to the same age as the duplicate column. Although the results thus obtained have value, it is of a secondary or substantiating nature. It seems unwise to include this group of columns when establishing a variation so intangible as that of column strength with length, which is already obscured by the natural large variation in strength of the concrete itself. Those columns having duplicate test cubes were also omitted, as they add another variable, the ratio of cylinder to cube strength.

The nine columns of *Bulletin 20,* University of Illinois, with cylinder strength graphically corrected* to the age of the columns when tested, and the three columns tested by Professor Berry,† all of approximately $\frac{L}{D}$ equal to 10.2 for the Illinois group, and 10.3 for the three Berry columns, give an average value for the ratio of column strength to cylinder strength, of 0.948, which agrees closely with the value 0.965 given by Equation (5) (graphically by Fig. 2).

Messrs. Buel and Reeves object to the statement that unreinforced concrete columns are not used in practice, yet they fail to substantiate their contention by citing a single instance of their use.

Concrete Columns Reinforced with Longitudinal Rods.—As additional proof that the longitudinal rods of a reinforced concrete column increase the strength of the column by approximately the yield-point stress per unit area of rod cross-section (as demonstrated on pages 1089-1092, inclusive, from a group of 12 selected columns), 24 columns of concrete of various strengths, with

* Taylor and Thompson, "Concrete, Plain and Reinforced", 1919 Edition, p. 320. The graph was the average of 19 series of tests.

† *Engineering Record,* February 19, 1910, p. 209.

from 0.99 to 2.9% of longitudinal reinforcement, described in "Tests of Metals and Other Materials" at Watertown Arsenal, 1904 to 1907, inclusive, have been analyzed. This group is not of the same usefulness nor weight as that of the 12 columns analyzed in the paper, as there are no duplicate test cylinders, and the strength of the concrete was determined by only 13 unreinforced columns of the same dimensions as the reinforced columns, a number of the unreinforced columns being tested at different ages from the comparative reinforced columns.

Applying these test results in Equation (18), using the ultimate strength of the comparative unreinforced concrete column corrected to the same age* as that of the reinforced column at test, the average stress in the rods at ultimate column stress is determined as 44 500 lb. per sq. in., a figure even more forcefully convincing than the value of 31 000 lb. per sq. in. given in the paper.

In order to discredit the theorem that longitudinal reinforcing rods increase the strength of a column by approximately the yield-point stress per unit area of rod, Mr. Godfrey states: "Of the rodded piers,† the group with 2% longitudinal steel is weaker than that with 1 per cent."

Mr. Godfrey evades the logical continuation and does not include the several columns unreinforced, and longitudinally reinforced with 4.075% of rods, because to do this would show that the average strength of the columns reinforced with any percentage of rods was increased at least by the approximate yield-point stress per unit area of longitudinal reinforcement. Thus, summarizing, we obtain:

Percentage of longitudinal reinforcement.	Average column strength, in pounds per square inch.
0	2745
1	3750
2	3690
4	4535

From this tabulation, or more vividly from the graph as given in Fig. 11† of the original report, it may be seen that, instead of the columns with 2% of rods being weaker than normal, the columns with 1% of rods were stronger than normal.

Strain-gauge measurements on the longitudinal rods, to determine the strength added by the longitudinal reinforcement, as suggested by Mr. Buel and by Mr. Ingberg, in showing a strain indicating a rod stress very near the yield-point stress just under the ultimate column load, have merit, but these measurements do not in themselves necessarily prove that the rods add strength to the column at the ultimate column strength, as the action of the rods would possibly, in some such unusual manner as that described by Mr. Godfrey on page 1152, in buckling out at a column stress of 150 lb. per sq. in. and breaking off the outer concrete, be the cause of the concrete failing at less than its uninfluenced ultimate strength. The strain measurements thus made on the

* Taylor and Thompson, "Concrete, Plain and Reinforced", 1919 Edition, p. 320.

† *Journal*, Am. Concrete Inst., February, 1915, p. 69.

longitudinal reinforcing rods are of very small magnitude, and are not an irrefutable proof that the stress corresponding to the strain in steel exists in the rods, because, for example, any slight distortion of the rods and the column proper from a straight line, or distortion of the rods within the column, would produce the same measured effect as the true strain; nor are longitudinal deformation measurements, unless made directly on the steel, positive proof of the measured strain, as concrete could possibly flow in such a manner that the strain in the rods would differ greatly from that measured longitudinally on the concrete of the column proper. The always at least partly unknown shrinkage effect also obscures the strain measurements, although any shrinkage would increase the stress in the rod reinforcement beyond that indicated by the strain measurements by putting the rods under initial compression. The accepted influence of the shrinkage of concrete on longitudinal reinforcing rods is without sound foundation. There is no reason to believe that, if the concrete shrinks, suffering a strain, e, and if E_c is the modulus of elasticity for that concrete, the stress necessary to restore that concrete to its original unshrunken length will be $p = eE_c$; nor, conversely, is it absolutely necessary that, if the unreinforced concrete would unrestrainedly shrink, suffering a strain, e, the concrete would produce a strain in embedded rods, the result of the action of the summation of the forces, $p = eE_c$.

The effect of the shrinkage of concrete on the strength of a longitudinally reinforced concrete column may be neglected, because this identical shrinkage occurred in the test columns, from which it was demonstrated that the strength of the column was equal to the maximum strength to be expected from the combination of steel and concrete. If shrinkage takes place and causes some compression in the longitudinal reinforcing rods, which is the probable action, then it follows that there is tension in the concrete. To obtain the concrete in the unstressed state, it would be necessary, therefore, to apply the load equal and opposite to the force (total rod area $\times$ stress) in the reinforcing rods. In order to cause the concrete to fail in compression, the load subjecting the concrete to its normal ultimate stress must be applied, in addition to the load necessary to overcome the initial tension stress in the concrete.

On page 1136, the writer states:

"The contention is untrue that, in the process of destruction of the [rodded] column, the rods buckle and cause the surrounding outer layer of concrete to scale, thus starting and aiding the failure of the column. The phenomenon is a phase in the failure of the testing of this type of column, and represents the buckling of the rods after the maximum strength of the column and the concrete has been passed and the concrete has broken, allowing the rods to buckle outward."

Mr. Godfrey, on page 1150, erroneously attributes this as the writer's deduction from the rapidity of failure in the longitudinally reinforced column. The statement is based on the study of tests on longitudinally reinforced concrete columns, and the impossibility, by reason of several obvious causes, of the rods causing column failure. Mr. Godfrey states that "* * * in many tests the rods buckle and push out the concrete. Simultaneously with this action, in a heavily loaded column, the center of the column would have to

crush." A "heavily loaded column" is evidently one loaded to the proximity of its ultimate strength, and "a heavily loaded column" in which the "rods buckle and push out the concrete" and "simultaneously with this action, the center of the column would have to crush", has reached the ultimate strength or load. Mr. Godfrey, therefore, admits that the ultimate column strength is reached "simultaneously" with the buckling of the rods, and, therefore, that the rods were not the cause of the failure of the column.

That the rods were not the cause of the failure of the column was presumed to be obvious. As contention has arisen, the reasons are herewith given:

1.—As shown by the 36 columns analyzed, the stress in the concrete of the longitudinally reinforced column at ultimate column load is equal to the ultimate stress of the concrete in the form of small test cylinders or unreinforced columns, and the stress in the reinforcing rods at ultimate column strength is approximately the yield-point stress for the steel of the rods. If the rods caused the failure of the column before the concrete had reached its ultimate strength, this could not be the case.

2.—The rods are stiffened and prevented from buckling by the surrounding concrete until the concrete shears at ultimate strain. The failure in concrete is in a shear plane inclined approximately 35° to the column axis. A rod passing through such a shear plane would still be stiffened by the remaining uninjured concrete, but the shear, moving the rod from a straight line, would put sufficient force on the restraining concrete to cause it to break off and permit the rod to buckle.

3.—By Professor Deimel's formula, it can be shown that if the length-diameter ratio of a steel rod is less than 40, it will not buckle before the yield-point stress is reached. The longitudinal distance between lateral restraint by ties is always less than this length-diameter ratio, hence the rod cannot buckle before the yield-point stress is reached. Moreover, the rod, surrounded by concrete, has no unrestrained length, and is held rigidly straight.

4.—For such small deflections from a straight line as exist in the reinforcing rods, it requires an enormous longitudinal force to cause them to buckle. For a longitudinal force producing the yield-point stress in the rods, the lateral force necessary to restrain the rod from buckling is very small by reason of the small lateral deflection of the reinforcing rod in the column, and the combined tension strength, bond strength, and shear strength of the concrete surrounding the rod, is overwhelmingly capable of supplying this small lateral restraining force, as can be shown by computation of these values.

Citing the condition of the Edison Building after the fire of 1915, Mr. Godfrey states that "many rods buckled out and pushed off the concrete, and the columns were still standing and carrying their loads. * * * This happened even in rooms where the heat was not sufficient to melt insulation on electric wires." He reasons that the "heat" was insufficient to cause the breaking off of the concrete, which must have been caused by the buckling of the rods, and that the load was not "heavy", because the uninjured concrete of the center of the column was sufficient to carry it. The evidence of the

unmelted insulation is of little value, as the kind of insulation is not known (vulcanized rubber, the possible component least resistant to the effect of heat, melts at 165° cent.), the possible difference in temperature between the wire insulation and the concrete of the columns having the outer concrete cracked off, due to the large size of the rooms of the Edison Building, and the fact that not a very high temperature, but, for concrete, a rapid change of temperature, is the far more actively destructive agent. Moreover, the heat causing such destruction, as described by Mr. Godfrey, could be supplied from another floor or room. In the report of the Committee on the Edison Fire,* there is the following statement:

"After the heat reached the reinforcement,† it was absorbed faster than the surrounding concrete, and the swelling of this metal, while not a primary cause of the damage to the concrete at the corners of the columns, did result in serious stripping of the concrete from the columns in many places."

It should also be noted that the longitudinal swelling of such heated rods would create considerable damage. That the particular columns were "still carrying their loads" can be seriously questioned. There is frequently small or no visible signs of failure in concrete strained just beyond its ultimate strain (approximately 0.0015), and the destruction of the outer surface of the columns is certain evidence of the indeterminate condition of the interior. Superficial observation is insufficient to determine that the column was still bearing load, as in the photograph, Fig. 13, as published in the report of the Committee on the Edison Fire,‡ it can be seen that at least four adjacent wall columns have completely failed, yet the building as a whole is self-supporting.

Fig. 56§ of the report shows a column such as that described by Mr. Godfrey, of which the center is apparently sound, whereas the outer layer is destroyed and the rods severely twisted and distorted in places, to a distance of a foot or more from the column. The title given to Fig. 56 is, as follows: "View of Interior Columns, First Floor, Building No. 11, Showing Exposed Reinforcement Due to Stripping of Concrete from the Corners of the Column Resulting from High and Rapid Changes in Temperature." It requires a vivid imagination to conceive a small strain, necessarily less than 0.0015, producing the buckling, twisting, and distortion to a distance of several feet from the column, which the rods are shown to have undergone. Of further interest and proof that the breaking off of the outer layer of concrete of the rod-reinforced columns was not due to load stress in the column, but to temperature, the following may be quoted from the same report:

"* * * it should be noted that the piers supporting the carpenter shop, which were not reinforced, were similarly spalled at the corners." (Page 612.)

* * * * * *

"There seems to be a general feeling that the damage to wall columns was the result of the application of water to the heated concrete." (Page 616.)

* *Journal*, Am. Concrete Inst., February, 1915, p. 116.

† The heat reached the reinforcement directly in many cases by means of expansion bolts in columns "for attaching piping, shafting, etc., to the structure", forming a direct heat contact to the reinforcement.

‡ *Journal*, Am. Concrete Inst., August, 1915, p. 599

§ *Loc. cit.*, p. 629.

"It was noted that in the beams and girders and frequently in the columns, expansion bolts or other metallic connections for attaching piping, shafting, etc., to the structure had been embedded in the concrete in such a manner as to form a direct heat contact to the reinforcement." (Page 623.)

It is not in itself convincing to offer authority in this form, but it is of at least as much weight as a statement made eight years after observations have been published.

Mr. Godfrey states that "by thus allocating any desired value to the strength of the concrete, of course, a reinforcing value of very definite amount can be found for upright rods, no matter what the strength of the column may show in the test." The value "allocated" to the strength of the concrete in the rod-reinforced concrete columns, in the analysis given in Section I of the paper, was not arbitrarily selected, but, for very good reasons, chosen as the strength of the concrete as exhibited in test cylinders. In the columns analyzed in this section of the discussion, the strength of the unreinforced concrete columns made of the same concrete as that in the rod-reinforced columns, was "allocated" as the value for the strength of the concrete in those rod-reinforced columns, both methods giving a logical value for the stress in the rods at ultimate column strength.

In his discussion, Mr. Godfrey repeatedly states that the rod-reinforced column is brittle, a quality which has "caused the failure of a large number of structures". He also states that "adding more brittle concrete to improperly designed [that is, rod-reinforced] columns, would have little if any benefit", not clearly indicating whether all concrete is brittle, or whether the simple addition of the concrete to a rod-reinforced column makes that concrete brittle. The Standard Dictionary defines brittle as: "Liable to break; fragile; frangible; fragile is synonymous with 'having little strength or endurance, frail, easily broken'."

There can be no question that in the general sense concrete is not brittle. The use of "brittle", and its antonym "tough", as given by Mr. Godfrey, terms for which there are no means of measurement, can lead nowhere. A scientific measure of the general vague properties of which brittle and tough are the extremes, is available in the ratios of the ultimate resilience per unit volume to the ultimate stress. The general ratio of resilience per unit volume to stress is of value, measuring the relative energy absorbed by different materials in producing equal stresses in them. For stress values up to the ultimate strain of unreinforced concrete, the ratio of resilience per unit volume to stress is practically equal for plain concrete, concrete reinforced with longitudinal rods only, and for spirally reinforced concrete.

Equal shock or impact energy is absorbed in producing equal stresses in the three types of structure. The energy necessary to produce ultimate stress and strain, however, is more than twice as great for the spiral column as for the rod-reinforced column. The difference, however, is not very great, and a long rod-reinforced column will absorb greater energy than a spirally reinforced column one-third as long, if both are of equal strength.

Concrete Columns Reinforced with Spiral.—The variations in the physical constants of concrete laterally restrained or unrestrained, even if the influence

and range of its non-homogeneity on these constants is large, have certain definite limits, and by the discovery of those limits, and the application of the maximum and minimum values possible to exist in rational theoretical formulas derived for a homogeneous, isotropic material, give as definite information as if applied to a perfect homogeneous and isotropic material. Substantiation of experimental observation by the results obtained by the application of theoretical equations, even when applied to non-isotropic concrete, gives added assurance.

The constancy of the modulus of elasticity (Young's modulus) within the range of application is not, as Mr. Ingberg states under the heading "Theoretical Consideration", necessary or essential for the validity of Equations (34) to (41) which express the internal equilibrium relations of the laterally restrained (spirally reinforced) concrete column. These equations connect the simultaneous values of a group of variables so that an identity results. It is admitted that the assumptions imply "values of modulus of elasticity * * * equal in all directions", as Mr. Ingberg states, an assumption not strictly correct, because the modulus of elasticity, like all the other physical properties of concrete, varies throughout the mass of the concrete. (See "Constants for Concrete", page 1079.)

The variations, however, between values for a point in the concrete along axes perpendicular to each other, will be within limits that are not very great, limits of variation to be expected closer than those of the strength of concrete. The lateral value of E_c and also $\frac{1}{m}$, effective in the equations, are average values for the whole horizontal section of the column at the point under consideration, and only a small variation in these total effective values is to be expected.

It is not true, as Mr. Ingberg states, that "the assumptions imply * * * a material able to resist up to the maximum all direct and induced compression, tension, and shear". The equations give absolutely no indication of compression, tension, or shear strength. They, of course, become inoperative and invalid when the concrete fails in any manner, specifically as failure actually occurs in the shear failure of the concrete of the core within the spiral, and throws a local stress on the spiral sufficient to cause tension failure.

The most important purpose of the equations of Section II of the paper is to prove that the failure of the concrete of the core is the cause of the failure of the spiral reinforced concrete column, one reason for this being the values of maximum strain in the spiral reinforcement at ultimate (maximum) column strength, as measured directly in the spiral, and as derived from longitudinal strain measurements in conjunction with theoretical considerations, and the application of the equations of Section II, are insufficient to cause failure of the spiral reinforcement by tension. The "contraction of volume [of the concrete] on drying out", which Mr. Feld states "directly concerns the failure of spirally reinforced columns", has "not been considered by the author." If lateral shrinkage occurs, it reduces the tensile strain and concomitant stress of the spiral at maximum column load, and, therefore, reduces the spiral stress values as determined in the paper, thus making it still

less possible that the spirals would be stressed to or beyond the ultimate strain and stress at ultimate column strength, the important finding of Section II. The effect of lateral shrinkage, however, cannot be considered, as knowledge of the magnitude of lateral shrinkage of concrete is completely lacking. The occurrence of longitudinal shrinkage has been firmly established by dependable tests, but the underlying laws of shrinkage are apparently yet unknown. A. T. Goldbeck, Assoc. M. Am. Soc. C. E., as quoted by Mr. Feld, on page 1161, obtained values of 0.0005 in. per in. for the longitudinal shrinkage on a 50-in. gauge length, but Professor Berry includes in his discussion of Mr. Goldbeck's paper, values of one-half this magnitude determined for the same length of time for a 20-in. gauge length. The size of the Goldbeck test specimen was 8 by 8 in. by 5 ft. The size of the Berry test specimen was 9 by 9 in., length not stated, but presumably approximately one-half the length of the Goldbeck test specimen. This suggests the very reasonable possibility of the shrinkage being a function of the length-diameter ratio, and that the phenomenon is an attempt of the concrete to assume a form of minimum superficial area, that is, every form of concrete would tend to assume the spherical form. There is no experimental evidence that the lateral shrinkage equals the longitudinal, or that lateral shrinkage occurs. It is possible that the lateral change in dimension would be an expansion.

Mr. Feld falls into several grave errors in attempting the computation of the stresses resulting from the contraction in "drying out". The contraction of 0.0005 in. per in. measured by Mr. Goldbeck, is linear, and if there is lateral contraction of equal magnitude, then the change of volume, or negative "dilatation" is 3×0.0005, or 0.0015 cu. in. per cu. in., with resulting values of normal stress of approximately 2 000 lb. per sq. in., and not 0.00017 cu. in. per in. and computed values of 450 to 650 lb. per sq. in. for the normal stress, as stated by Mr. Feld. The normal stress, discussed by Mr. Feld, is the stress over the total superficial area, necessary to produce the negative "dilatation", σ. It is totally unrelated, however, to the stresses produced in either the concrete or the spiral reinforcement, because if a prism, column, or other form of unreinforced concrete, shrinks unrestrainedly, no stress is produced in that concrete. If steel reinforcement was embedded in concrete that has shrunken, the stress produced in that steel would be measured neither by the negative dilatation of the unreinforced concrete nor the normal stress necessary to produce compression in the concrete equal to the shrinkage that has taken place, but would be a function of the linear contraction or strain as determined by Messrs. Goldbeck and Berry, and also dependent on the relative ratio of steel and concrete. The stress produced in the steel by shrinkage, provided, as stated under "Rod Reinforced Columns", that the concrete after shrinking follows the equation, $p = eE_c$, can be readily shown to be given by the equation:

$$P = \frac{E_s S}{n \frac{r}{1 - r} + 1} \qquad \text{.....(D)}$$

in which,

p = the stress produced in the reinforcing steel by the shrinkage, S;
n = the ratio of the modulus of elasticity of the steel and the concrete;
r = the ratio of reinforcement; and,
E_s = the modulus of elasticity for steel = 30 000 000.

A shrinkage of 0.0005 will produce a stress of 12 500 lb. per sq. in. in 2% reinforcement.

Method of Failure of Spirally Reinforced Concrete Column.—That the failure of the core concrete is the method of failure of the spirally reinforced concrete column, at the same instant maximum column load is sustained, and that the actual failure of the spirals in tension is a subsequent phenomena without importance, seems to be established. In a few instances it appears that the failure of the core concrete occurred in such a manner that the spirals were capable of restraining the broken core, resulting in comparatively large longitudinal ultimate strain, with a slowly increasing stress, with increase in strain after the core had failed. This method, however, is unusual, and the small increase in stress is of no value; a small number of repeated stresses of the magnitude causing the core failure would cause column failure.

As Mr. Ingberg states, the stress-strain graph for undrawn steel given in Fig. 10 is very different from that for drawn steel. However, as steels of both kinds were used in the spiral reinforcement of the test columns, with elongations of 1.5% in 8 in. for one specimen,* and upward, the independence of the strength added to the concrete by the spiral reinforcement and the strength of the reinforcement and also the manner of fabricating the steel is established. In drawn steel there is no yield point, the yield point having been raised to coincide with the ultimate stress. Small as the minimum ultimate elongation of 0.015 is, it is greater than the maximum possible lateral strain exhibited by spirally reinforced columns. As 0.5 is the greatest possible value of Poisson's ratio, a lateral strain of 0.015 requires a longitudinal strain of at least 0.03. This is true for an unbroken core concrete. If the core failed in shear, it is possible, but not probable, that sufficient local stress would be induced, in the spiral at the shear plane, to cause spiral tension failure through a longitudinal strain of less than 0.03 in. per in. Even for a collection of separate (but not separated) masses, the state of the core concrete of the spirally reinforced column, after a strain of 0.0015 is reached, as suggested by Mr. Feld, on page 1163, or, of a very large number of separate particles, such as laterally restrained sand, the measurements corresponding to Poisson's ratio, if of sufficient number to give a true mean value, would have the same theoretical upper limit of 0.5 and could not have a larger value, because of the direct application of the principle of the conservation of energy.

Mr. Feld's theory of the method of failure of spirally reinforced concrete columns can be experimentally demonstrated to be incorrect. He contends that "at an axial strain of 0.0015, the concrete has developed its maximum stress and fails; that is, the adhesion is broken", and gives further statements

* See Column Nos. 30, 32, 33, Table 6.

of "separated surfaces" and "the incipient separation of the concrete at a strain of 0.0015." (Mr. Feld also gives a maximum ultimate strain of 0.00142, as determined by Walker,* for a corresponding stress of 5 000 lb. per sq. in. This figure is, of course, in error; the two stress-strain diagrams included in the American Concrete Institute tests of 1915,† show strains under the ultimate of 0.0017 + and 0.0015 + for stresses of 2 750 and 2 950 lb. per sq. in.)

That this concept of a broken mass of concrete is incorrect, can be disproved as follows:

1.—Mr. Feld's hypothesis can be interpreted only by the complete separation of the concrete at the planes of shear, simple at first, but with continued compression, extending throughout the mass, as inferred by him. If the concrete removed from a column that has been strained well beyond 0.0015 in. per in. shows no indication, by test, of being broken into separate masses by that strain, the hypothesis must be either false or meaningless. Mr. P. H. Bates who was in charge of the Pittsburgh Branch of the U. S. Bureau of Standards during the two important series of tests for the American Concrete Institute in 1915, and for L. J. Mensch, M. Am. Soc. C. E., in 1916, is very well informed on both series of tests. After the von Emperger columns were tested, the cast-iron reinforcement was dug out of the columns. As Mr. Mensch states, the final strain put on the columns was very large, at least 0.006 in. per in., or four times the ultimate strain of unreinforced concrete. The whole column was then strained well beyond the value of 0.0015, and if Mr. Feld's hypothesis is true, the concrete within the spirals should be a broken mass of concrete. However, for points other than within a foot or so of the actual failure, this was not the case; severe blows with a heavy maul were required to break the concrete. In a letter on this subject to the writer, Mr. Bates states

"* * * to all intents and purposes * * * the concrete away from the failing portions is uninjured. * * * if the spiral were removed, the concrete would not fall apart, in fact, it would require considerable effort to break it apart * * * this concrete, however, if tested in the form of small specimens as an 8 by 16 in. cylinder, would probably show that it had been rather much overloaded."

Mr. Bates is reasonably correct in that the apparently uninjured concrete of the failed column, if tested unrestrainedly in test cylinders, would show reduced strength, a result in accord with general knowledge already gained of the result of fatigue.

2.—The result of a test made under the direction of M. Considère,‡ is also of substantial evidence, although it was unfortunately conducted (as is too often the case with concrete investigation) with only one specimen. Briefly described, a spirally reinforced test column of concrete strength, as indicated by an unrestrained concrete column stress of 80.3 kg. per sq. cm., was stressed to 666.7 kg. per sq. cm. The spiral was then stripped in the middle of the column for a length of 3 diameters, and the load was again applied, necessitating 61.2 kg. per sq. cm. for failure. This is proof sufficient that the concrete

* *Bulletin 5*, Structural Materials Laboratory, 1920.

† *Journal*, Am. Concrete Inst., February, 1915, p. 65.

‡ "Rapports Relatives à l'Emploi du Béton Armé", 1907, pp. 323–324.

was not badly cracked before the second loading, and that beyond the ultimate strain of unreinforced concrete, the core concrete is still whole and uninjured until the ultimate, or maximum, column stress is attained.

3.—The reinforcing spiral is strained to approximately 10 000 lb. per sq. in. when the column load stresses the concrete to its unreinforced ultimate strength, varying with the ultimate strength of the concrete, but in all cases to a point well below the yield-point stress of the steel. If shear failure in the core concrete begins at the strain of 0.0015 in. per in., and increases with increasing strain, the increase in stress and strain, of concrete spirally reinforced, is due solely to the restraining action of the spiral on the broken concrete within the spiral, and must be entirely independent of the original strength of the unrestrained concrete. There is, however, experimental contradictory evidence in the two columns tested by Professor Withey* and described in Table 6. These spirally reinforced columns, of concrete of the high strength of approximately 4 900 lb. per sq. in., undergoes but little increase in strength due to the spiral reinforcement, and that increase is due to the restraining action of the spiral at the ultimate stress and strain of the unreinforced concrete. The stress-strain diagrams for these columns show maximum or ultimate column load to be carried at approximately the strain of the unreinforced concrete, and, from observation, Professor Withey stated, as quoted on page 1115, that the failure of these columns was due to the failure of the concrete. After the failure of the concrete, with stresses in the spiral approximately the same as those for lower strength concrete at the same column strain, the spiral is unable to restrain the concrete and produce any increase in load. The large column stress (4 900 lb. per sq. in.) cannot be the reason for this inability of the spiral to increase the load, as spiral reinforcement in columns of lower strength concrete is capable of producing loads greater than this in lower strength concrete.

The Increase in Concrete Strength Due to Spiral.—The ultimate strength of spirally reinforced concrete was expressed by Equation (21):

$$f'_{cs} = (1 + kr_s) f'_c$$

in which,

f'_{cs} = the ultimate reinforced column stress;
r_s = the ratio of spiral reinforcement; and
f'_c = the ultimate strength of identical concrete as exhibited in test cylinders.

k connects the three other variables.

From the group of 39 columns analyzed in the paper, it was apparent that, for a range in concrete strength between 1 150 and 4 000 lb. per sq. in., k was a constant having the value of 0.573. The value of k was apparently also independent of the yield point (when present), strength, and ultimate strength of the spiral reinforcement.

In Table 29, Mr. Mensch gives data from European sources, which, if operative for materials used in the United States, tend to modify these conclusions.

* *Bulletin 466*, Eng. Series, Univ. of Wisconsin.

Mr. Mensch is unfortunate in selecting the 10 columns tested by Professor Bach (Nos. 17 to 26 of Table 29) as proof of the very large value of k for low strength concrete. These tests, for several reasons, give very little information of value. The length-core-diameter ratio of the test specimens is 1.9—certainly very much too short for a column. The columns are of two groups: One group of a 1:2:3 concrete giving a 28-day ultimate stress of 1 850 lb. per sq. in.; the other not a concrete, but a 1:5:0 mortar, giving a 28-day ultimate stress of 875 lb. per sq. in. Mortar is impractical as a column constituent because of the difficulty of mixing. It is also uneconomical, as for the Bach tests, the same percentage of cement in mortar gives only 46% of the strength it develops in the concrete, and even with the much greater percentage increase in strength of the mortar, due to spiral reinforcement, still develops less total strength than the concrete when both are equally reinforced.

There are, however, other experimental evidences of the proportionately much greater strength added to "lean" concrete,* which indicate very large increase in strength due to spiral reinforcement of approximately 1 200 lb. per sq. in. concrete.

The group of seven columns tested by Professor Rudeloff† and presented by Mr. Mensch are by no means conclusive proof of the variation of the increase in strength of spirally reinforced concrete with variation in elastic limit values of the spiral reinforcement. Their length-core-diameter ratio of 4.6 is very short. The three groups of plain concrete columns tested during the series, of which the tests in question were one group, and which should, because of conditions as constant in fabrication and proportioning as those which exist in the group of columns tested to determine the influence of the variation of spiral strength, show no more variation in strength than this group of columns, give average strengths of 174 000, 176 000 and 156 000, the three columns prepared by Professor Rudeloff for comparison with the group discussed having strengths of 155 400, 167 400 and 145 800 kg. The strength of the concrete in the reinforced columns is, therefore, not as constant, nor as reliable as the strengths of the complementary test cubes indicate. Two groups of test columns having the same reinforcement and same strength of concrete, described in the paper by Professor Rudeloff, mentioned previously, which can be compared, therefore, with the seven columns tested expressly to determine the effect of variation of the strength of the steel on the strength of the reinforced concrete, show a decrease in strength with increase in the spiral strength. Indicating these two new groups with a star(*), we obtain the results given in the following tabulation:

Yield point, in kilogrammes per square centimeter.	Total column load, in kilogrammes.
2870*	209 330*
3220	205 300
3930*	198 400*
5920	267 500
6710	296 700

* "Rapports Relatives à l'Emploi du Béton Armé", 1907.

† *Deutscher Ausschuss für Eisenbeton*, Heft 25, 1913.

The group of columns* tested by Professor Talbot give results directly contradicting the conclusions of the very large increase of strength of lean concrete due to spiral reinforcement and of the variation in the strength of spirally reinforced concrete with spiral strength. These columns, which are given in Table 6, are numbered 6, 7, 8, 9, 10, 29, 30, 31, 32, and 33.

The information of present interest is summarized in Table 36.

TABLE 36.

Column No.	Spiral.		Strength of concrete, in pounds per square inch.	Ratio of column strength to cylinder strength.	Value of k.*
	Yield point stress, in pounds per square inch.	Ultimate stress, in pounds per square inch.			
6	60 000	90 000	1 650	1.56	66
7				1.47	55
8	38 000	53 000	1 433	1.54	63
9				1.47	55
10				1.53	62
29	54 000	73 000	1 150	1.81	50
30	115 000	152 000	1 900	2.00	60
31	54 000	78 000	1 900	1.99	60
32	115 000	152 000	1 800	1.89	52
33	115 000	152 000	1 400	1.94	54

* In Equation (21).

In Table 6, Columns 6 and 7, having reinforcement with a yield point of 60 000 lb. per sq. in., show no greater increase in strength than Columns 8, 9, and 10, having reinforcement with a yield point of 38 000 lb. per sq. in. Columns 30 and 31 of concrete of the same strength have the same increase in strength from spirals with yield points of 54 000 and 115 000 lb. per sq. in., respectively. Columns 29 and 31, which have spiral with a yield point of 54 000 lb. per sq. in., have exactly the same value for k (that is, percentage increase in concrete strength due to spiral reinforcement as Columns 30, 32, and 33, which have spiral with the yield point of 115 000 lb. per sq. in. Column 31, with 1 900 lb. per sq. in. concrete, has a greater value for k than Column 29, with 1 150 lb. per sq. in. concrete. The value of k for Column 29 of very low strength concrete is slightly below the average value, instead of three times the value for medium strength concrete, as indicated by the European tests.

The results of the European tests may be due to some peculiar property of the cement, sand, or other variable, or it may be that the true action of lean concrete and of variation in column strength due to variation in strength of spiral reinforcement in the American tests has been hidden by a combination of unfortunate circumstances. If the results as obtained for the increase in strength of lean concrete reinforced with spiral are general in application, then k will be constant and equal to 0.573 for values of concrete strengths between 1 700 and 4 000 lb. per sq. in., approximately, which is the range limit for the strength of concrete for which spiral reinforcement is used. The formulas given in the paper for the ultimate strength and for the safe working strength of spirally reinforced concrete columns are, therefore, valid for the strength

* Described in *Bulletin No. 20*, Eng. Series, Univ. of Illinois.

range of concrete used in practice, and on the side of safety for very lean concrete, provided the large increase in strength due to spiral as obtained by M. Considère and Professor Rudeloff can be duplicated in materials used in the United States.

The strength of the spirally reinforced concrete column, as Mr. Godfrey states, is a function of the pitch (spacing) of the spiral reinforcement, the percentage of reinforcement remaining constant. The pitch of the spirals in the columns analyzed in Section I is "about 1 in.", to quote Mr. Godfrey, and the column spirals, representative of the usual practice, have a pitch of about one-tenth the core diameter. Within the limits of pitch to column diameter found in practice, and typified by the group of test columns, the variation in column strength due to variation in the ratio of spiral pitch to core diameter is negligibly small.

Mr. Godfrey analyzes the internal reactions of the spirally reinforced column, as follows:

"In a hooped column, the succeeding hoops make of the column a system of superimposed flat disks. It is well known that a flat disk of any material, such as concrete, is of very much greater strength than a cube or a cylinder two diameters in height. A cube of lime mortar will stand very little compression, but a thin joint of the same mortar will stand a heavy pressure, and the thinner the joint the greater its carrying capacity. This effect is independent of the lateral confinement, except the confinement that is afforded by the friction of the end planes. In the hooped column, the hoops offer a corresponding confinement. In the spiral reinforced column, a somewhat similar action takes place, and the pitch of the spiral is of great importance."

The analysis given by Mr. Godfrey is completely in error. The core concrete is not divided into flat disks by the spiral, disks which have the same strength in the "hooped" column as they would have if tested individually in the testing machine. The determining factor for the strength of flat disks in the testing machine is, as with the cube, "the result of the method of compression failure of the concrete * * * the cube form forcing the failure to occur", instead of in the unrestrained plane inclined at about 35° to the axis, "in a plane inclined at an angle of 45° or more." This preventative action of the plattens of the testing machine increases the force necessary to cause failure. The failure of the core concrete is a shear failure*: The hoops do not divide the core into horizontal sections through which the shear failure cannot pass. The shear plane, being free to select its own angle of inclination with the axis of the column, extends a longitudinal distance greater than the core diameter, and, consequently, through a number of the "flat disks." As stated by the writer, the "method of failure must be understood for the further analysis of reinforced concrete elements."

Mr. Reeves quotes the results of the series of nine columns tested by Professor Berry,† as evidence that spiral reinforcement does not appreciably increase the strength of the column, and, quoting Mr. Godfrey, "the results of tests on * * * plain and reinforced concrete columns" * * * shows

* See pp. 1113-1117, inclusive.

† *Engineering Record*, February 19, 1910, p. 206.

remarkable agreement in one thing; they begin to fail at nearly the same unit load, no matter what the reinforcement or lack of it." The reason for the small increase in strength of the spirally reinforced columns tested by Professor Berry (through no fault of Professor Berry) was the poor combination of spiral in a square column. The minimum of concrete protecting the spiral was 1 in. If the same concrete area (of the 14 by 14-in. square column) was in the form of a round column, and spiral, 2 in. less in diameter, of the same percentage was used as reinforcement, the total column load would have been 32% greater, or 48% greater than the strength of the plain concrete columns.

Mr. Godfrey states that column failure "begins at practically the same unit load, no matter what the reinforcement." Is a hardly noticeable cracking of the protective shell occurring at much less than ultimate column strength as important as the cracking of the concrete columns without spiral reinforcement, which is the almost instantaneous indication of ultimate column strength and simultaneous column failure?

The group of six columns (M-1, M-2, M-3, and M-14, M-15, and M-16, Table 6), are dangerous by reason of the large addition to strength which they indicate for 0.5% of reinforcement. The statement in the paper cannot be controverted:

"Even if these two groups of columns of high strength can be shown, in the future, to have possessed this high strength due to some property of the concrete, the spirals, or the method of treatment of the concrete, etc., until this strength can be shown capable of reproduction at will, the reason for the high strength must be considered as accidental and to have no bearing in the derivation of allowable working stresses."

The group, M-14, M-15, and M-16, is of such short length-diameter ratio (4.3) as to give little foundation for the action of a column of usably large length-diameter ratio and can be omitted on that ground alone. The group, M-1, M-2, M-3, in showing such a much greater relative increase in strength than the M-17, M-18, M-19 group, is sufficient to make the M-1, M-2, M-3 results questionable.

The designation of different weights to the results obtained from various investigations is a firmly established custom in analysis of scientific material. The weight to be assigned to the results of the McKibben-Merrill tests* from inspection of the variation in strength of the concrete both in test cylinders and in plain columns, in comparison with the evenness of strengths obtained by Professors Talbot and Withey, and in the American Concrete Institute tests, can be seen to be much less than that to be assigned to these latter three. It seems probable that the strength of the concrete of the different batches of the McKibben-Merrill tests varied considerably; the strength of the plain concrete columns cannot then be used as an indication of the strength of the concrete in the reinforced columns. The variation in the test cylinder strength, the only other means of determining the strength of the concrete in the reinforced columns, is so large as to make results derived on this basis very insecure.

* *Proceedings*, Am. Concrete Inst., February, 1915.

The two groups of columns testing high cannot be used in safe stress determination. They serve no other purpose than the possible indication of a very high strength due to unknown causes. As these causes are unknown, they cannot be applied to obtain these high strengths in actual practice. Had the group of columns tested low the same amount as they tested high, then, certainly, they would have to be included in determining their variation and resultant reliability number, N_r.

The von Emperger Type Column.—With Mr. Mensch, the writer feels that the safe stresses indicated in Section IV, Sub-Section D, of the paper are lower than the true safe stress, but consistency with the disparagement of sentiment in the derivation of safe stresses and elsewhere in the paper, demands that this intuition, as a reason for increasing the suggested safe stress values for the von Emperger type column, be ignored. Theoretical consideration apparently indicates that the spirally reinforced concrete surrounding any defective and low-strength part of the cast-iron core would not prevent those parts failing at the stress at which they would fail if unembedded. It is true that, as Mr. Mensch states, "the probability that the weakest part of the hooped concrete surrounds the weakest part of the cast iron is extremely small"; but this is not the pertinent question. Failures of all the columns of this type were failures first in the concrete. If the individual cast-iron reinforcement was defective so that, unsurrounded by spirally reinforced concrete, it would fail at a stress less than that imposed in it at a strain equal to the ultimate strain of the reinforced concrete surrounding it, would that surrounding concrete prevent the cast-iron failure? There is no experimental data to prove that it would, and, consequently, no increase in working stress should be given the cast iron above that permitted for the unembedded cast-iron reinforcing element. As the type of column indicates the possibility of very high working stresses, it is to be hoped that this experimental support will be forthcoming; the method of obtaining it would require very careful consideration.

Mr. Mensch, surprised at the ultimate strain value of 0.0034 found in the paper, gives as ultimate strain values for the von Emperger type "0.006, the average of all columns measured was 0.0082, and the ultimate strain * * * probably exceeded 0.01 in. per in." How these can be the correct values is to be questioned, for, using the average stress strain curves for the three cast-iron test specimens in the group, the average strain of 0.0082 in. per in. given by Mr. Mensch would indicate a corresponding cast-iron stress of 53 000 lb. per sq. in. The average load of the nine similar von Emperger columns, less the total load in the cast-iron reinforcement at a stress of 53 000 lb. per sq. in., shows a stress in the spirally reinforced concrete at maximum column load of only 3 400 lb. per sq. in., contrasting with the average ultimate stress in the test cylinders of 4 120 lb. per sq. in., and stresses of 4 580 and 4 970 lb. per sq. in., respectively, in the columns reinforced only with spiral and rods.

The best method of determining the ultimate strength of the concrete in the group of von Emperger columns, and on which the determination of the stress in the cast-iron reinforcement at maximum column strength depends, is open to question. Mr. Mensch prefers the unit strength of the columns

reinforced with rods and spiral only, to the strength based on the test cylinder. He offers as the explanation of poorly made cylinders, "that the workmen were accustomed to tamping columns, but not to making test cylinders"; but the strength of the cylinders relative to the columns was greater than normal. The writer prefers the evidence of the twelve cylinders to that of the two columns reinforced only with rods and spiral.

The Endurance Limit of Concrete.—The endurance repetition load limit of concrete, as determined by Professor Van Ornum,* and discussed by the writer in Section IV of the paper, coincides of necessity, because the strain also approaches an asymptotic value, with the time endurance load, a point of importance omitted in the paper.

Mr. Ingberg, on the basis of endurance experiments conducted by W. K. Hatt,† M. Am. Soc. C. E., and Professor Berry,‡ questions the reduction (50%) due to repetitive stress, as determined by Professor Van Ornum, as too large.

The tests by Professor Hatt on beams, with 1776 and 1248 repetitions, and, for the remainder, 710 or less, are a very small number indeed, and measure the endurance of the steel reinforcement, rather than that of the concrete.

Even if the stress in the beams tested by Professor Hatt is a larger proportion of the ultimate strength of the concrete than that obtained by Professor Van Ornum, such stress, on account of the "continually increasing deflection and set under this process of repeated loading," cannot be used as a safe working stress, even if the "maximum load which the beams carried" was "not diminished by the repetitions of load," because the increasing deflection and set would cause failure.

The tests§ by Professor Berry do not contradict, but tend to support and extend Professor Van Ornum's test results. Professor Berry tested the beams with from 920 000 to 1 890 000 repetitions of a stress not greater than 40% of the ultimate and probably considerably less than that for most of the test specimens. Professor Berry finds that the "repetition does not materially affect the ultimate strength, the position of the neutral axis, the maximum deflection, nor the ultimate deformation in the concrete," conclusions all of which are in agreement with extension of those of Professor Van Ornum from his test results, that is, for repeated stresses with values below the endurance limit of concrete, the ultimate strength and strain of the concrete are not changed. The endurance test results of one cylinder, as given by Professor Berry in 1910, can have only very small value, as the one time loading strength of the cylinder was unknown.

Reliability Determinations.—Mr. Cox, in his excellent discussion, has supplied the frequency curves of actual tests, as suggested by Mr. Mensch. The exact form of the curve cannot be determined graphically because of the small number of concrete test columns available, and "the necessity for an

* *Transactions*, Am. Soc. C. E., Vol. LI (1903), p. 443, and Vol. LVIII (1907), p. 294

† *Proceedings*, Am. Soc. for Testing Materials, Vol. 7 (1907), p. 421.

‡ *Loc. cit.*, Vol. 8 (1908); Vol. 9 (1909), p. 493; and Vol. 10 (1910), p. 581.

§ Described in *Proceedings*, Am. Soc. for Testing Materials, 1903 and 1909.

enormous number of observations in order closely to approximate the theoretical curve." Even with a much larger number of test specimens (Nos. 76 and 183, respectively), as available for the curves given in Figs. 14 and 15 by Mr. Cox, the exact form of the curve indicated by the test points (or areas) is very questionable. The form of the "normal" frequency curve can be determined with far more accuracy from Equation (47).

The method of determining the equation for the skew as well as the form of normal curve, by means of the duration curve, given in the pertinent and useful discussion by Mr. Foster, and in his paper on "Theoretical Frequency Curves", previously mentioned, gives apparently the best means of determining the skew of the curve as well as the form. The number of test results is so meager, however, that the amount of skew for the probability curves for the various types of columns could not be determined.

Safe Working Stresses.—Both Mr. Foster and Mr. Cox, in their discussions, present interesting and important ideas on safe stresses. The economic factor presented by Mr. Cox is worthy of further study and promises to be of service. There can be, however, but one true safe stress for a material or structural element, which can be determined by methods indicated in the paper, a value not to be exceeded when human safety depends on the structure. For special cases, other values for the variables, N_r and L_e, of Equation (54), than those given in the paper, can be determined. A temporary structure does not require a value of L_e as low as 0.50, and for the structure the failure of which does not endanger human life, the value of N_r may be appropriately chosen by the proper balance of the saving in cost of material, possibility of failure, and loss in case of failure. Such processes of determination are impractical; the solution appears to be in the determination of the true safe stress and the introduction of additional "economic" multiplying factors for various conditions of service and types of structures.

Mr. Godfrey, on page 1152, questions what is to be done if a batch of bad concrete is placed in a rodded column, or some concrete is affected by frost (?), and solves the problem by "exert [ing] himself to make his structures 'fool-proof', not only by using a factor of safety to cover contingencies" (contingencies, by the way, causing stresses of entirely unknown magnitude) "but also by using a design that will produce tough columns." Mr. Godfrey, of course, metaphysically transmutes the batch of bad concrete placed in a spirally reinforced column. Such transmutation, however, does not affect concrete, and "bad" concrete is incorrigible, regardless of where it is placed. The writer recognizes "the impossibility of assuring a safe value," a safe value being something that "no mathematical equation can assure," as Mr. Godfrey evidently is unaware when he questions the procedure when "concrete is affected by frost." The writer's suggestion of assuring a minimum strength concrete "by taking check test specimens from every batch of concrete" is a certain solution of the problem, whereas the proper factor of safety to cover the loss of concrete strength due to freezing is difficult to determine. The possibility of loss of concrete strength by freezing is slight, as has been determined by pyrometer temperature readings in the surface of newly poured concrete.

By applying Mr. Godfrey's definition of "toughness" as "the quality of standing overload without distress," one can make plain concrete or any material as "tough" as desired by assuming a working stress as small as necessary. The basis from which "overload" is measured must be the permissible working stress, itself heretofore a point having a vague relation to ultimate stress. The ratio of percentage overload to cause failure is nearly equal in the rod and spirally reinforced columns, hence, according to Mr. Godfrey, they would be equally tough. If he considers the cracking of the protective shell of spirally reinforced concrete, however, as signs of distress, then the rod-reinforced column is tougher than the spirally reinforced column.

Mr. Godfrey admits "uncertainties in execution and unknowables in design," and talks of a "margin" to cover these uncertainties and unknowables. Mr. Godfrey's method of determination of the magnitude of the "margin", a perfectly definite value, to cover what is, to Mr. Godfrey, uncertain and unknowable, would be of interest. In his paper, the writer presented and applied to some extent methods of determining maximum values and eliminating "unknowables" and "uncertainties." The actual stress in a structure is unimportant, but the maximum possible value is all important.

The statement that "a unit volume of concrete has the same ultimate strength no matter where placed" is admittedly ambiguous. The writer wishes to call Mr. Godfrey's attention to the fact that "a unit volume of concrete" surrounded by spiral is a "unit volume of spirally reinforced concrete." The intention was to call attention to the fact that, for example, in a reinforced beam, because the concrete in the middle of the top fiber fails first, it is not the weakest, but the stress at that point is greatest.

The several apparently contradictory parts of the paper, to which Mr. Godfrey calls attention, will be clear if the paper as a whole is carefully read.

Appreciation.—The writer desires to express his appreciation of the discussions of the paper, especially those parts in which an endeavor was made to extend the principles developed in it. He desires to thank the Society for the privilege of presenting the paper and publishing it, and especially to thank the Secretary and his staff for their courtesy and patience and sustained interest in producing the paper.

The writer wishes also to acknowledge his debt to Mr. P. H. Bates, of the U. S. Bureau of Standards, by whom he was first inspired with an appreciation of experimental engineering research, and who introduced to him the problem treated in the paper.

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Paper No. 1524

ENGINEERING EDUCATION

TECHNICAL PAPERS PRESENTED AT A JOINT SESSION WITH THE SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION AT THE ANNUAL MEETING, JANUARY 17, 1923

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THE NEW PROJECT FOR THE SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION*

By Charles F. Scott,† Esq.

At the Annual Convention of the Society for the Promotion of Engineering Education, in June, 1922, the Council appointed a Development Committee to formulate an answer to the question, "What can the Society do in a comprehensive way to develop, broaden, and enrich engineering education?"

The following is quoted from the report of the Committee:

"The engineer of to-morrow will be called upon to take even a more important part than the engineer of to-day in great enterprises, both private and public. He will be called upon for general service in organization and leadership as well as for technical service in the solution of new problems (industrial, economic, governmental and social) which the engineering applications of science have produced. This means imagination, vision, the power to conceive, and the ability to carry out great projects.

"The problem of engineering education is to determine and to meet the progressive demands of a rapidly changing civilization. What constitutes these demands is a question being considered by educators, by engineering societies, by organizations of industries and public utilities and by others. Unfortunately, there has been almost no co-ordination of effort, and there is lack of agreement on even fundamental objectives. The first necessity then would seem to be an investigation to determine just what the facts are."

* * * * * * *

"The question involves two fundamental inquiries:

"A.—What the engineer graduate should be.

"B.—What the training of the engineering student should be and the influence surrounding him.

"Your Committee would therefore recommend:

"I. That there be created within the Society, a Board of Investigation and Co-ordination, consisting of five (5) members, under whose general direction there should be organized and conducted (*a*) an active campaign for the promotion of engineering education in the light of the needs of the future, as those needs may be developed; (*b*) there shall be co-ordinated as far as possible the activities of the various agencies interested in promoting engineering education; and (*c*) there shall be conducted research in engineering education."

* * * * * * *

"III. That there be authorized a director, with an adequate staff.

"IV. That progress reports to the Society be published periodically."

* * * * * * *

"VI. That the objects (among others) for which the Board is created shall be considered to be:

"1. To ascertain the facts in engineering education, such as concerns (*a*) teachers, their origin, training, experience, and effectiveness; (*b*) teaching facilities; (*c*) curriculums; (*d*) students and graduates, their origin, training, experience and effectiveness.

"2. To ascertain present and future requirements in the fields served by engineer graduates.

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* Presented at a Joint Session with the Society for the Promotion of Engineering Education at the Annual Meeting, January 17, 1923.

† Pres., Soc. for the Promotion of Eng. Education; Prof. of Elec. Eng., Sheffield Scientific School, Yale Univ., New Haven, Conn.

"3. To present the facts and requirements for their bearing on the training of the engineer to the end that he may (*a*) develop himself and his profession; (*b*) realize and fulfill his obligation to society.

"4. To maintain close contact with engineering schools enabling them to participate in the investigations, and reporting to them from time to time, to the end that the developments may be continuous from the initial contact between the colleges and the agencies of the Board.

"5. To secure the necessary funds for these purposes."

The report has been approved and the members of the Committee have been appointed as members of the Board. They are as follows: M. E. Cooley, M. Am. Soc. C. E., Dean, Colleges of Engineering and Architecture, University of Michigan; John H. Dunlap, M. Am. Soc. C. E., formerly Professor of Hydraulics and Sanitary Engineering, State University of Iowa, now Secretary of the American Society of Civil Engineers; Dugald C. Jackson, M. Am. Soc. C. E., Professor of Electrical Engineering, Massachusetts Institute of Technology; F. W. McNair, President, Michigan School of Mines; and the speaker. F. L. Bishop, Secretary of the Society for the Promotion of Engineering Education, is Secretary of the Board.

On behalf of the Board, a letter was sent to presidents and deans of engineering colleges asking suggestions and co-operation by the schools. The responses were encouraging and constructive.

The full functioning of the Board must await the securing of funds and the selection and appointment of a Director and staff.

The increasing general interest in engineering education is significant. The National Industrial Conference Board and the Society for the Promotion of Engineering Education have recently formed an Advisory Conference Committee on Engineering Education. The members representing industry are: S. P. Bush, President, Buckeye Steel Castings Company, Columbus, Ohio; H. E. Coffin, Vice-President, Hudson Motor Car Company, Detroit, Mich.; Col. T. C. Dickson, U. S. A., Commanding Officer, U. S. Arsenal, Watertown, Mass.; Howard Elliott, Affiliate, Am. Soc. C. E., Chairman, Northern Pacific Railroad, New York City; E. M. Herr, President, Westinghouse Electric and Manufacturing Company, East Pittsburgh, Pa.; William H. Nichols, Chairman, Allied Chemical and Dye Corporation, New York City; A. H. Rogers, President, Mining and Metallurgical Society of America, New York City; and, Henry D. Sharpe, Treasurer, Brown and Sharpe Manufacturing Company, Providence, R. I.

The members representing the educational society are as follows: R. H. Fernald, Professor of Mechanical Engineering, University of Pennsylvania, Philadelphia, Pa.; H. J. Hughes, M. Am. Soc. C. E., Dean, Harvard Engineering School, Cambridge, Mass.; D. C. Jackson, Professor of Electrical Engineering, Massachusetts Institute of Technology, Cambridge, Mass.; F. W. McNair, President, Michigan School of Mines, Houghton, Mich.; J. W. Roe, Professor of Administrative Engineering, New York University, New York City; H. Schneider, Assoc. M. Am. Soc. C. E., Dean, College of Engineering and Commerce, University of Cincinnati, Cincinnati, Ohio; and the speaker.

It was a surprise to the Committee to find practical unanimity in the view that engineering education for the industries should be along broad lines,

with thorough training in the fundamentals of science and engineering, leaving much of the training in the special processes to experience to be gained from the industry after graduation. It was the general view that the narrowly or specially trained man was not fitted for development into the positions of larger responsibility and leadership which industry needs; that the industries at present are not appreciative to what a degree their development and expansion is dependent on securing men of engineering training and giving them opportunities for development. It is anticipated that the Industrial Conference Board not only will awaken industry to a new attitude toward science and engineering, but that it will be most helpful to the engineering schools by giving a clearer specification as to the type and number of men which industry needs, and will assist by co-operating with the schools in making their training most effective.

The Highway Education Board is significant as a forward movement. Leaders in highway transport are making education a prime factor in the program for the development of highway transport. One is tempted to ask whether the railroads would be in their present position, if they had cultivated education in past years and had grafted into their organizations generous numbers of men of scientific and engineering training. Certainly, conditions would not be as they are at present, in which there is little demand or opportunity for college graduates in railway service, and research in the railway field is practically at a standstill.

Engineering and scientific societies have been taking a new interest in engineering education. The American Institute of Mining and Metallurgical Engineers and the American Chemical Society have already issued reports on education. Dean Cooley who recently visited engineering societies and engineering schools in a third of the States in his travels as President of The Federated American Engineering Societies, and Secretary Dunlap who made many visits in his journey to and from the Fall Meeting of the American Society of Civil Engineers, in San Francisco, Calif., both report a general and animated interest in engineering education.

The Presidential Address to the American Society of Mechanical Engineers by Dean Kimball in December, 1922, was entitled "National Leadership". In this address, Dean Kimball showed that our civilization differs from those that have gone before in our power to produce the necessities of life, to feed, clothe, and house the multitude. He traced the changes wrought by modern methods on industrial life and the resulting complexity of human relations. Modern civilization is largely what the engineer has made it, and the civilization of the future will be largely what he wishes it to be. Modern civilization, however, is not what it should be—produetion has increased, but adequate social benefit has not resulted. The engineer for the first time is face to face with the great problems of human relations in industry and the distribution of wealth. A new form of industrial leadership is needed, but the source of this leadership does not seem to be clear. In history, National leadership was first a military rule, succeeded by legal government. Industry has had these types, and two other types are available for industrial leadership, namely, the busi-

ness and financial type and the scientific type. Dean Kimball showed why and how the engineering type of mind is adapted to make a great contribution to the solution of industrial and National problems.

Most engineers will agree with Dean Kimball that the extending field of the engineer must logically include, not only the technical and the strictly professional, but also participation in leadership in industry and in public affairs.

Industrial, social, economic, political, National, and international problems are largely those which are incident to the new world brought about by science and engineering. The opportunities, the responsibilities, and the obligations of the engineer to-day differ from those of fifty years ago in proportion as the field and influence of engineering itself has grown; and the opportunity and responsibility of the engineer in the next generation can be predicted only as we project into the future the curves of engineering and related activities and attempt to formulate the various problems which the next generation will bring forth.

Engineers themselves have realized a field of service beyond that of their old-time technical societies and have formed The Federated American Engineering Societies which is making available the service of the engineer in a new way.

Anticipating that the engineer of the future must be bigger and broader than the engineer of to-day, it becomes necessary to provide the new engineer. This is the function of the engineering school, but not of the school alone. Engineers and engineering societies, industry, and others as well, should consider what the engineer of the future should be, what type of boys should be chosen, and what education and training they should receive in school and after graduation, in order that the engineer of the future may be adequate for his new responsibilities.

The Engineering Profession has been unique in having a National organization for the promotion of engineering education. The Society was formed at the Columbian Exposition in 1893. Through conventions, by papers and discussions, by reports of standing committees of which there are a score, and by its publications, it has promoted the ideals and the practice in the training of engineers. Through its initiative, the report of Dr. Mann of the Carnegie Foundation for the Advancement of Teaching was produced, which has awakened thought and discussion. The new project of the Society has been undertaken in recognition of the new demands on engineering schools. It is not a plan by which some super-committee shall evolve a formula for engineering education. It rather contemplates active interest and participation by the schools in formulating and solving the problems which are primarily their problems.

The schools are alert to new methods. Co-operation with industry, administrative engineering courses, discussions regarding the lengthening of the course from four to five or six years, and the constant agitation for change in curricula indicate the desire for something different. The new fact-finding and co-ordinating Board established by the Society aims to aid and direct progress

by co-ordinating the ideas of the various schools with those of industry and of the Engineering Profession.

It is not anticipated that a single general type of engineering school will be found adequate or desirable. Presumably, objectives and ideals will be more clearly defined, and each school will find, by self-determination, its own particular field. Its industrial environment, its type of students, its educational relations—whether part of a university or an independent school—its financial resources, its physical facilities, and its teaching equipment are all factors determining the service it can best render.

There are certain broad underlying questions in regard to which the investigations should furnish data, among which are the following:

What class of subjects will best supply the cultural broadening outlook which technical courses lack?

Is the broader attitude of the student to be acquired through the non-technical so-called cultural subjects or through engineering teachers who have a larger conception of the relations of the engineer to society?

What differentiation should be made in the training of different types of men and in training for different fields of service?

Are the present faculties composed of men who have the technical knowledge and practical experience and teaching ability and the attitude toward life which adequately fit them for training the engineers of the future?

If not, what constitutes the ideal teacher and what measures should be taken to secure or to train engineering teachers?

Engineering societies can aid in specifying the ideal teacher, and can assist in finding him and inducing him to be a teacher. The engineering society may have more weight than the engineering faculty in giving boards of trustees a true vision of the relation of the engineer to society, which may result in wise selection and compensation of engineering teachers.

The rising cost of education in common schools, high schools, and colleges is discussed in the last report of the President of the Carnegie Foundation for Teaching. A large part of the great increase is attributed to new ideals regarding education, for example, that the school and college form a ready path to ease and prosperity with minimum effort, that every field of human knowledge must be included in the curriculum, and that vocational training, even if it is superficial, must be given by the school. The engineering school is subject to the same tendencies, and a keen study of its objectives and purposes is a first essential in determining what it should do and the most efficient way of doing it.

The whole field is so large that the new Board must content itself with the initial selection of a few definite topics. The curriculum has been suggested as such a topic, not as a mere list of topics to embellish a page of the catalogue, but rather as the subjects which give character and content to engineering education. The first essential is a definite determination of the product the curriculum aims to produce. Furthermore, how is the curriculum to be administered? What type of teacher and what facilities are requisite? Broadly considered, a curriculum study may involve directly or indirectly many

of the apparently diversified questions of engineering education. An outstanding feature of the speaker's recent contacts with teachers, engineers, industrialists, and fathers of boys is the serious importance with which engineering education is discussed. It is not mere academic discussion. In a recent conversation regarding the new project of the Society for the Promotion of Engineering Education, the speaker stated that detailed plans had not been made, as a large part of the task is to formulate the problem, but what gives him inspiration and confidence is the large vision, the serious purpose, and the enthusiasm of the men of varied experience who constitute the new Board.

The Board is appreciative of this opportunity for a joint session of its Society with the American Society of Civil Engineers. It is most gratifying that the oldest of the National engineering societies is taking a part in engineering education. The weight of its counsel and advice will be welcomed by the Society for the Promotion of Engineering Education at large and by the engineering schools individually. The American Society of Civil Engineers has had a leading and conspicuous part in the engineering achievements of the past and present and now has opportunity to be an important factor in shaping and developing the engineers of the next generation. What is the vision of the Civil Engineer and the American Society of Civil Engineers as to the duties the engineer should assume in the next generation? This is a question of profound consequence to civil engineers, to all engineers, and to society at large.

THE OUTLOOK FOR THE ENGINEERING SCHOOLS OF THE MIDDLE WEST*

By William G. Raymond,† M. Am. Soc. C. E.

The speaker recently addressed a questionnaire to the deans of the engineering schools of the Middle West, including two foundation schools, the Northwestern and Washington Universities, and sixteen publicly supported schools. The questions were as follows:

1.—What has been your growth in total attendance in professional engineering courses for the first semester of the school years 1920-21, 1921-22, 1922-23?

2.—What has been the growth of the freshman class for the same period?

3.—Are the space, equipment, and teaching staff maintained in entirely satisfactory ratio to the attendance, or is either one of these items failing to grow sufficiently to give wholly satisfactory results?

4.—Is the preparation of entering students as a rule wholly satisfactory, or is there a noticeable deficiency in one or more lines, and, if so, in what line?

5.—Has the dropping of solid geometry and third semester algebra from general college entrance requirements produced any difficulty in handling first-year students presenting themselves for admission? If so, can you suggest a remedy?

Replies were received from the two foundation schools and twelve public schools.

Questions 1 and 2 were designed to secure replies that would indicate the outlook for attendance. Unfortunately, many people measure the worth of a school by the number of its students.

The Middle West is an agricultural district, and the farmer has been hard hit in this post-war period. His products were the first to drop in price to a pre-war level, and the prices of what he consumes have remained at a high level. As a result, the prediction was made in 1920 that college attendance in the Middle West would show a marked decrease, and that, perhaps, this would be particularly true of the attendance at engineering schools. The prediction has proved to be unwarranted. Most of the colleges are crowded beyond capacity with respect to space, equipment, and teaching staff.

The increase began prior to 1920, and the engineering schools and departments generally were crowded like the other branches. This was thought to be due to the advertising of the profession by its wonderful war service; but whatever the reason, the attendance at the engineering schools of the Middle West is greater than ever before, except during the first part of 1918-19, although it was a little less in 1922-23 than in 1921-22. The total attendance in the 14 schools answering the questionnaire for the beginning of the years given, was as follows:

1920-21	1921-22	1922-23
11 285	12 274	11 872

* Presented at a Joint Session with the Society for the Promotion of Engineering Education at the Annual Meeting, January 17, 1923.

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The attendance in one of the largest schools is not included. The total attendance at the 18 schools is estimated to be about 15 000.

Perhaps of more significance is the registration of freshmen, as this is a better index of what the attendance of the next few years will be. The registration of freshmen in the 14 schools making returns has been:

1920-21	1921-22	1922-23
4 609	4 505	4 057

Therefore, although there was a larger total enrollment in 1921-22 than in 1920-21, and also a larger total enrollment in 1922-23 than in 1920-21, there was a decrease in freshman enrollment in both years, the enrollment in 1922-23 being nearly 12% less than that of 1920-21.

The peak permanent freshman class was that of 1919-20, when many high school students of 1917 and 1918, who had been in the Service, increased the regular class of 1919. This class will disappear in June, 1923, and, thereafter, a marked decrease in total attendance may be expected as a result of this disappearance and the presumable continued decrease in the entering class. Not all schools show a decrease in the entering class. In Iowa, both the schools have had a larger freshman class in 1922-23 than in 1921-22.

It may be said with some certainty that the immediate outlook for the engineering schools of the West, is for a smaller attendance. This outlook will not be displeasing to the directing officers of the schools, as in the majority of schools there will be relief from overcrowded conditions. Only four schools, two of which are foundation schools, reported adequate teaching staffs, space, and equipment.

The immediate outlook for maintenance of staff, space, and equipment in these agricultural States, the chief industry of which has been in great distress, is not good. It will be some time, apparently, before the schools reach an efficient basis, unless the combined effect of some additions to staff, space, and equipment, and a considerable reduction in attendance brings about that condition of balance toward which they are all striving.

The outlook with respect to character of work done in these Western schools is very good. Administration officers and faculties are studying contents of curricula and methods of teaching, and are inquiring into the needs of the profession with respect to two matters: First, the demand the employer with reason expects to make on the new graduate; and, second, the demand the public is making and is going to make in the future on the trained engineer of experience.

Perhaps it is beside the purpose of this paper to indicate what those who have made such inquiries have discovered or think they have discovered, but the risk will be taken and the statement made. It is believed to have been demonstrated by the replies to questionnaires sent to employers, by conversations with employers, and by printed statements that have appeared occasionally that the demand of the employer is for men thoroughly educated in the fundamental principles of engineering, and trained to reason correctly when using these principles. Employers seem to be aware that engineering schools are not trade schools, nor schools for the teaching of the details of the art of

engineering, such as schools of dentistry and medicine, which teach the details of the art of the practice as well as the science. The art of engineering is so extensive and varied and is changing so constantly in many branches that any person, whether he is a teacher or an employer, who is familiar with the work of the engineer must realize that, in four years of college work, it is impossible to teach both the fundamental theory on which the practicing engineer must rely for the development of his art, and also the details of that art, which to a considerable degree vary with each particular commission in which such art must be applied, if indeed it can be taught at all in college. Colleges of dentistry and medicine have their clinics and hospital practice, law schools their practice Courts, but engineering schools cannot build bridges, locomotives, railroad track, generators, telephone lines, etc., for the purpose of student practice.

The inquiring faculties think they have discovered, also, that the engineer is rapidly coming into what may be called "his own"; that the public is recognizing more and more the value of his advice on public enterprises—often not purely engineering enterprises. It is recognized that his training —narrow as it has been—and his professional experience, when it has been somewhat broader than his training, has fitted him peculiarly to reason correctly on economic questions as well as on questions of the technical application of the laws of physics, mathematics, and chemistry; and it seems to be more and more desirable that in college the young engineer should be trained somewhat in the fundamental principles of economics and accounting, and should have brought to his attention somewhat of the results of historic events.

The discovery or development of these things and one other circumstance have brought about the movement among the mid-Western schools for a more extended college training, "for those engineering students whose aim is to become qualified to take positions among the creative leaders in the profession."

The other circumstance is the poor preparation of the graduates of secondary schools in the Middle West. Possibly, this poor preparation is not peculiar to that territory, but certainly it is in evidence there whether or not it is in other parts of the United States. Almost all the engineering schools of the mid-Western States are supported by public funds, either State or Federal, or both, and the State University or the State College of Agriculture and Mechanic Arts is looked on as the head of the educational system of the State. It is expected, therefore, that these institutions will receive into their freshmen classes the graduates of any approved, four-year, high school, just as the high schools receive those who have finished the grammar schools, and the grammar schools receive those who have finished the elementary grades. Approval of a high school seems often to depend not so much on the character of the work done in it, as on the number of rooms, the number of teachers in proportion to students, and other considerations of this kind. Moreover, following the example of the colleges, there has been a distinct tendency in the high schools away from education and toward instruction, the difference between which terms has been defined recently.

In the mid-Western States there is an organization known as the North Central Association of Colleges and Secondary Schools, which defines subjects

acceptable for college entrance. The list has become so long that it is easy to find the necessary fifteen units without using either quadratic equations in algebra, solid geometry, or language other than English. There is in the State of Iowa, and perhaps in other States, an organization known as the Board on Secondary School Relations. This Board has abandoned what is commonly known as the third half year of algebra, which included quadratic equations, solid geometry, and languages other than English, as essential college entrance requirements. In this decision, the Board may have been acting under a real or imaginary compulsion, as have some of the colleges in dropping these subjects. There is a tendency to try to make the secondary schools practical schools, supposedly training the greater number of their students for business life, because only the smaller proportion of their students expect to go to college. As the more advanced algebra and solid geometry have been abandoned as essential college entrance requirements, many of the high schools do not offer these subjects, and students who expect to go to engineering schools, all of which require these subjects as essentials to entrance to the first year, are not informed that they must cover these subjects, nor are they given opportunity to do so. They present themselves for admission to the engineering schools and are turned away or entered at a disadvantage, various unsatisfactory arrangements being made for their progress. Few of them complete their work in four years.

There seems to be a distinct tendency away from subjects that seem to be difficult, and from examinations to test the student's knowledge. It is somewhat difficult to locate the responsibility for these tendencies, and the speaker will not attempt to do so lest, in his ignorance and poor judgment, he should receive the condemnation of teachers' associations, and colleges and departments of education, which, it seems to him, have been yielding to a popular demand growing out of hastily formed erroneous ideas of the function of the school. Whatever the cause may be, however, it is certain that graduates of the average four-year high school of the mid-West are not prepared as they should be to enter a college of engineering that is attempting to give its students a proper training for their profession in four years.

The speaker does not know why the four-year college course was adopted rather than a three-year or a five-year course; but, at one time, when fewer principles of engineering practice were known, and when graduates of reputable high schools were better prepared in the fundamentals of mathematics, English, and other languages, than they seem to be at present, possibly due to the fact that there were fewer graduates from these schools, and only those who were especially adapted to scholastic work, four years of college training in engineering may have been sufficient. In those days, a course included more of general educational subjects such as history, foreign language, literature, and economics, than it does at present in the Middle West.

The members of the Society for the Promotion of Engineering Education know, and, perhaps, most of those who may read this paper know, that, having in mind all the points that have been mentioned, the deans of fourteen of the engineering schools of the Middle West assembled in Chicago, Ill., in May, 1922, to consider what should be done, particularly with respect to the length

of the college course in engineering. After discussing the matter for a day, they adopted a set of resolutions which, amended at a subsequent meeting in Urbana, Ill., are, as follows:

"The undersigned engineering deans, directors and representatives, in conference assembled, hereby resolve that,

"In order to meet the constantly enlarging responsibilities of the engineering profession, we favor an advance in engineering education at this time that shall provide five years of collegiate training for those engineering students whose aim is to become qualified to take positions among the creative leaders in the profession, and that such advance shall be made in substantial accordance with the following plan:

"1. Include in the four-year engineering curricula a substantial proportion of fundamental and humanistic subjects, omitting, if necessary, a sufficient amount of the more advanced technical work. It is desirable that, so far as possible, the curricula in the different branches of engineering shall be sufficiently uniform to permit students to defer their final choice of a specialty at least to the end of the second year.

"2. Add a fifth year of advanced work, mostly or wholly technical, and specialized to such an extent as desired.

"3. The first four years of work shall lead to a bachelor's degree and the fifth year to an advance degree in engineering."

Re-assembling in June, 1922, at the time of the meeting of the Society for the Promotion of Engineering Education in Urbana, and after having been considered by the several faculties concerned in their respective schools, the resolutions, with a slight and unimportant amendment, were re-adopted as given and stand as the combined judgment of the immediate administrative officers of all these schools.

To resolve is one thing; to do is another; and in each school there are distinct difficulties in proceeding in accordance with the resolutions. There would be no difficulty except a financial one if all the schools would unite on an essentially common program, but each school is more or less afraid of the effect on its attendance of the adoption, by it alone, of a college course of five years. Unfortunate experience has been had by three of the mid-Western colleges in this respect. One of these is a foundation school and the other two are State supported schools.

It is believed, however, that the deans of these mid-Western schools are correct when they conclude that the broad, professional engineer, competent to design and to direct engineering enterprises of magnitude and to take his proper place in public affairs, cannot acquire the training in fundamental technical principles and an adequate knowledge of what may be called social science, even for a beginning in his life work, in four years of college training, after graduation from the average mid-Western secondary school.

There is some question as to how the five years or more shall be divided, whether two years shall be devoted to ordinary arts training, followed by three years or more of technical training in the engineering school, or whether the entire five years or more shall be within the engineering school, social and technical engineering subjects being carried side by side throughout the course with, perhaps, the final year devoted more particularly to the technical work of the engineer.

It is believed that the prevailing idea has been expressed by F. E. Turneaure, M. Am. Soc. C. E., in a letter* which states exactly the plan that is being considered most favorably by the faculty of the school with which the speaker is connected, and is thought to be that in the minds of most of the administrative officers who joined in the discussion at the meeting held in Chicago, in May, 1922.

Dean Turneaure's statement of the plan and the advantages that he sees for it are, as follows:

"In brief, this would consider a four-year engineering course (or pre-engineering course) which would be very general in its character. It would contain a large amount of work of general educational value, such as history, economics, language, science, chemistry, physics, mathematics, and mechanics, also the elements of applied work in the usual branches of engineering. Such a course I would consider the minimum requirement for any young man who desires to enter the industries or any field of engineering whatever; and for many lines of work it would be a satisfactory preparation. It would also furnish a sufficient foundation in mathematics and science to enable a man of real ability to advance himself in almost any line of engineering activity, even where a longer course of training would be preferable. By the elimination or postponement of a very considerable amount of technical studies now included in the regular four-year course, such a course as that indicated could be made quite satisfactory from the standpoint of general education, and, on the whole, much more satisfactory than a four-year course in a college of liberal arts, such as would be elected by students planning on an engineering course later. At least, it would meet the needs of a great many more students, and I am inclined to think that it would be the best course for practically all.

"The four-year course as indicated should lead to a degree (such as the B. S. degree), without special designation. Then those students who desire a more thorough engineering education should secure it by further study, either in the form of graduate work or fifth and sixth-year courses leading to an appropriate second degree. These advanced courses would be well organized, and would be taken by a very considerable number of students, as they would contain a good share of the material now included in our senior year. They would naturally be taken by all students desiring to enter the research field and those aiming to become fairly well posted in any special line. I should suppose that from 25 to 40% of those finishing the four-year course would continue for a fifth or sixth year, the percentage depending, of course, quite largely upon the curriculum adopted.

"Among the advantages of such a scheme as above suggested are possibly the following:

"1. It requires all engineering students to secure a much broader general education than is now the case, while at the same time it includes a sufficient amount of fundamental technical work to satisfy the requirements in many cases.

"2. It furnishes the well-organized technical work for special and advanced students which will encourage students of ability to thoroughly prepare themselves for research and special work of various kinds. Graduate courses in engineering will be much better organized than is now the case.

"3. It is a better arrangement than that frequently proposed of requiring a two- or three- or four-year college course before admission to the engineering school. In any pre-engineering course, a considerable amount of mathematics and science must be required in order to save time, and pre-engineering courses

* *Engineering Education*, December, 1922.

under the control and advice of the engineering faculty can be better adjusted to the students' need than when otherwise administered.

"4. It would, I believe, solve the problem of the four-year course, against which so much criticism is directed.

"5. It would tend to eliminate from the more advanced and technical work students who are unfitted for such work while at the same time capable of becoming successful engineers in certain lines of employment. With the very great numbers now attending engineering schools, a relief of work in the senior and junior laboratories would be very acceptable in most cases."

It is believed that the attitude of the schools of the mid-West is shown in the preceding discussion and indicates an extremely good outlook for the character of the work to be done, in reading the signs of the times aright, and in adapting the work to changing standards and requirements for the trained engineer.

Engineering research is being developed with considerable rapidity in a great many schools. There are those who say that the primary function of a university is research, that its teaching work is incidental. It is believed that, except with respect to those institutions created primarily for research, such a statement is wide of the fact, and, with respect to the State universities of the Western and Southern States, it is certainly not in accordance with the ideas in the minds of those who created those institutions.

Standing as the capstones of the educational systems of the States, the State universities were created primarily to give what was commonly known as a college education to the youth of those States. Research was hardly thought of by any one connected with the institutions when they were created. In connection with these institutions, this idea is of modern growth.

Research is an important function of a university, and it may be, but is not necessarily, an important function of an engineering school. It is believed to be fair to state, however, that those schools in which engineering research of importance is being developed, are schools in which the students even though they do not participate in research work, are brought into an atmosphere that is wholly beneficial; and it is also fair to state that, in the engineering schools in the West, research is being developed with some rapidity.

It has been impossible for the speaker to learn and record, with any accurate detail, the research activities of the several schools; but many are probably familiar with certain outstanding work accomplished at some of the schools, such as the work of the great Forest Products Laboratory at one school; the experiments in sanitation and the investigation of numerous railroad, mechanical, and fuel questions in another institution; investigation of the strength of drainage tile and pressure of earth in trenches on such tile, and certain questions of interest to highway engineers, in another institution; and elaborate, large scale experiments to determine the extent to which the hydraulic jump may be utilized, at still another institution, favored with a river of considerable size flowing through its campus, are all reasonably well known to the Engineering Profession. These activities in engineering research are positive indications of the advancement of the engineering school faculties of the West. As a result of the development of research work, there

is a considerable registration of graduate students engaged in advance work related to this research.

The proposal that has been made that these institutions correlate their work so that each will engage in those lines of research for which it may be peculiarly adapted, or prepared already, and duplication of effort avoided, would seem to be another indication of advancement.

On the whole, therefore, a consideration of what has been said would seem to warrant the conclusion that the outlook for the engineering schools of the Middle West is an extremely favorable one; that the ratio of equipment and teaching staff to attendance—although at present unfavorable—is tending to become more and more favorable by the reduced attendance and increased facilities; that the character of the work done in the schools, always good, is constantly improving; that they are adapting themselves to changing standards and requirements for the trained engineer; and that they are developing with some rapidity that important function of research, which is enabling them to add in no small degree to the already accumulated store of professional knowledge.

CO-OPERATION OF NATIONAL ENGINEERING SOCIETIES IN ENGINEERING EDUCATION*

By John L. Harrington,† M. Am. Soc. C. E.

Engineering education and the possibilities of co-operation by the engineering societies in the solution of its problems is too broad a subject for rounded discussion in a brief time. Therefore, consideration will be limited to one phase only, namely, the direct relation of the National Engineering Societies to the students as it finds expression in the establishment and conduct of Student Branches.

The practicing engineer has always considered it to be his function to point out the shortcomings of the engineering schools and to criticise the curriculum, the work of the faculty, and the delinquencies of the students. As an employer of the product of the schools, his assumption of this function is not without warrant, and in so far as his criticism has been constructive, it has been helpful; but too commonly the practitioner has looked on the student, not as his successor, but as his tool, and has decried his imperfections and scolded the faculty for permitting them. He has demanded a finished product, men thoroughly trained and ready to do work of high commercial value, men of direct and immediate service to him.

The engineering student has commonly held somewhat the same view. He has demanded strictly technical, directly useful knowledge and has been impatient with the collateral subjects essential to the broad foundation required for ultimate achievement of a high character. As new engineering schools were established and competition for students was increased, it is small wonder that the faculty bowed to the demand of the student and the practitioner alike and filled the curriculum with technical studies to the exclusion of all but the most essential preparatory subjects.

The result has been that the schools have turned out men of increasingly narrow training as specialized knowledge accumulated. Forty or fifty years ago, civil engineering included all but military engineering, and technical subjects were so few and so little developed that many academic subjects were necessarily included to fill out the year's course generally considered essential for sufficient mental training. For many years, however, the number of technical subjects and the demand for them have been so great that the curriculum has been divided into civil, mechanical, electrical, and chemical engineering, and even these branches have many sub-divisions, so that a graduate has not only little cultural education, but is without satisfactory knowledge of the fundamental sciences which support the technical work.

In like manner, engineers engaged in certain branches of technical work became impatient with the necessity for considering the problems of those engaged in other lines, developed a high degree of pride in their specialty, and organized separate societies of increasingly narrow purposes. Often the objects of the society were so restricted that the number of its members was

* Presented at a Joint Session with the Society for the Promotion of Engineering Education at the Annual Meeting, January 17, 1923.

† Cons. Engr., Kansas City, Mo.

insufficient for effective operation; and as it is exceedingly difficult to make distinct divisions, the activities of many organizations overlap. This has compelled the individual to maintain membership in several societies in order to sustain close contact with all his interests.

In recent years, as the professional engineer has become dissatisfied with his place in the world and has come to resent the fame and fortune accruing to the lawyer, the promoter, or the business man who made use of his technical knowledge while he remained in the background, he has slackened his demand for strictly technical training and is now urging more education in those branches which will enable him to make complete use of his technical knowledge, to build on it administrative and executive functions which formerly were considered beyond his province. This breadth of view, however, is so recently established that his earlier habit of segregation and specialization still dominates his mind to such an extent that he fosters the interests of his special group to the hurt of his profession.

Increasing professional consciousness has steadily increased interest in the training for the profession, and the engineering societies, stimulated by their members in the universities, have not only concerned themselves with the character of the curricula of the engineering schools, but have sought by establishing student branches to help the student directly to an understanding of the work, the ethics, and the ideals of the engineer, and to further his development after graduation by leading him to join the society devoted to his specialty. These purposes, both worthy, are not entirely altruistic, for the societies profit by the readier transition on graduation from membership in the student branch to membership in the society; but to the credit of the societies it must be admitted that this purpose is secondary.

The best of purposes are, however, sometimes mistaken. In considering the benefits, the evils escape attention. The operation is successful, but the patient remains an invalid or even dies. So it is in this instance.

A thorough survey of the condition of the student branches made in 1922 by Dr. William H. Kennerson, Dean of the School of Engineering of Brown University, in the course of which he visited branches in all parts of the United States, compelled him to conclude that most student branches were sustained by artificial respiration administered by interested members of the faculty. His report discourages the thought that the student branch is accomplishing the purposes for which it was formed, and an analysis of the matter will clearly disclose the reasons.

In all co-operative effort the benefits received are substantially proportional to the efforts put forth, and as the student branches are organized by the society and kept alive by the faculty, the student's activities are limited and his benefits are proportionately small. In some cases, competent student leaders secure worthy results, but, generally, the student is already surfeited with the leadership and direction of others and his interest is small where the initiative and control are not his own.

Not only are the student branches failing measurably to serve the purpose for which they were organized, but they are doing their members a great injury. It is generally agreed that the training of the engineer is much too

narrow, that in addition to a certain amount of technical knowledge, supported only by the absolutely essential mathematics and the barest elements of the directly related sciences which now constitute his training, it should include a superior command of English and of at least one other language, sounder training in chemistry, metallurgy, and physics, and a substantial knowledge of history, literature, logic, psychology, economics, accounting, geology, mineralogy, and other subjects essential to the support of the technical work and to the broad training of a truly professional man. It is well understood that the engineer especially needs training in the science of politics, in understanding, organizing, and controlling men; yet, in establishing and fostering student branches, each little group is set off by itself, and its interests are restricted to only one branch of engineering and to its own members.

The eminent engineer with the broad message of interest is no longer invited to deliver it to all the students of an engineering school, but to one division of them, to a student branch; or, if his message is technical, only those studying his specialty hear it. This results in greater difficulty in securing the interest of able men who are unwilling to take the time to serve a minor group, and, manifestly, it results in restricting the knowledge and narrowing the vision of the students.

Both the students and the societies would be better served, and energy, effort, and funds would be conserved, if the four Founder Societies could agree on a plan of cordial co-operation in educating and training the future members of the profession. The Honorary Engineering Society, Tau Beta Pi, offers an excellent example for a society composed of engineering students of all kinds, having a chapter in each of the engineering schools and supervised by a council of four, one to be elected each year by the delegates from the chapters in convention. Each member of the council should serve through four years and should be selected rotatively from lists submitted to the convention by the respective National Societies. Each student chapter should be advised, but not governed, by suitable members of the faculty. The actual management should be in the hands of the students who would have the initiative and who would thus be trained for future activities in the professional societies. The close association in each chapter of students of all branches of engineering, the acquaintance of all with the publications of all the Founder Societies, the meeting of all together when an address is to be delivered, the presence of all when any engineering matter is under discussion, and the interest in the chapter of all members of the engineering faculty, will broaden the student's knowledge of the profession, will teach him to work in harmony with those whose interests and views differ from his own, and will strengthen him in every way for the battle which follows graduation. Institutions having one or more groups too small to be organized into a student branch may well secure a chapter of such a co-operative society to the distinct benefit of both the students and of the Founder Societies, and each student will become so well acquainted with every branch of the profession that he will choose from among them with greater certainty of his fitness for the work of his choice. Every one is familiar with failures resulting from errors

in selecting a vocation arising from lack of knowledge of other vocations. The drafting rooms of the larger engineering works contain many technically trained men who have failed to accomplish according to their abilities, because of error in the choice of work. Selection of a specialty to-day is largely a matter of whim or chance, not of knowledge of the field.

A recent writer discussing the purposes of the students has stated:

"I have known hundreds, thousands of undergraduates, but I cannot think of one who has a clear idea of why he has come to college. I hasten to make two exceptions. Engineering students know that they come to learn to be engineers, but they know that they must learn something more than that—and they do not know what that extra something is."

What are the National Societies doing to help solve the problem? Are their divided and inharmonious endeavors helping? Does it broaden the student, make him wiser and more tolerant, to see his elders in the profession carrying their differences of organization and opinion down into the kindergarten of professional training?

There is no substantial reason, no reason that a broad-visioned man can reasonably give, why the Founder Societies cannot whole-heartedly co-operate in carrying to the student the lesson and the support they have for him. For several years, they have been making an energetic endeavor to co-operate in rendering disinterested service to the nation, in bringing to bear on the public affairs in which they are peculiarly fitted to serve, the united strength of the profession. Such large bodies move slowly, but with great effect once they are in motion; but before they can materially swerve the views of others, they must show their own ability to accept the truth, to sacrifice something of the individual interest for the general good. Let us stop the discord at its source and bring the engineers of the next generation under greater harmony of thought and action than we have been able to achieve, by uniting in a common effort to broaden and train them for a unified service to the profession and to the world.

THE OBJECTIVE IN ENGINEERING EDUCATION*

By Magnus W. Alexander,† Esq.

The ultimate aim of education, especially in a democracy, must be the advancement of the moral, intellectual, and physical standard of all the people for the benefit of all the people. The Greek philosopher, Plato, formulated the general concept by stating: "Good education is that which gives to the body and to the soul all the perfections of which they are capable." Modern democracy, however, makes even this formulation broader by emphasizing service to society as the basis of Plato's conception.

The objective of engineering education is to provide society with men who have had that general education so aptly formulated by the Greek philosopher and, in addition, have been trained in a knowledge and understanding of the exact sciences through the applications of which they are enabled to promote the physical well-being of the community. By their habits of exact thinking and constructive outlook, engineers are particularly able to serve as leaders in public and private affairs, because, guided by truths already discovered, they look forward to find safe and sound footings for new structures, rather than look backward at the cold stones of precedent. This pioneer spirit, well grounded and intelligently directed, is the great asset of the engineer.

The pioneer is invariably one who has discovered that his existing circumstances or environment do not permit what he considers the full use of his powers. Whether he is actuated solely by a selfish motive or whether, like Galileo, he is moved by a desire to share with others the fruits of his own thoughts and labors, he finds himself bound by barriers which must be broken down. Strangely enough, these barriers of tradition and inertia are often most strongly supported by those who should be the first to welcome progress. Consider the opposition and hostility to those who sought to give steam navigation to the world. We now stand amazed and amused at the scientists who only a century ago tried to prove mathematically the impossibility of a ship crossing the Atlantic by steam power. Only because men with true engineering spirit, convinced by the results of their painstaking experiments, would not be turned aside from their search for the power which would carry them against adverse winds and currents, can we, to-day, board a palatial steamer with the conviction that we will be carried to our destination, almost regardless of wind or weather, and will arrive on schedule time.

Let this, as one of many similar examples that could be cited, suffice to sustain the thought that, in our efforts to build a better and truer engineering education, and, for that matter, a truer education of general or specific character, we must develop a spirit of tolerance under which to lessen, even if we may not entirely remove, unreasonable opposition to rational effort for progress.

What is needed to-day in a society that has grown more and more complex in its organization and the relations of which are growing more and

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more interdependent, is exact, clear thinking on a basis of ascertained fact and experience. It is not enough that the groundwork should appear substantial; it must extend to the bed-rock of truth. How often has quicksand been found in the construction of great railroads or in the erection of lofty buildings? The fact that, in former years, these sands had been considered an insurmountable obstacle to such construction did not daunt the spirit of those who had been taught to search deeper than the shifting sands. By the aid of their science, they froze the sands and made them temporarily useful, while they dug through and beneath them to place solid footings on the unyielding rock.

By establishing the social structure on a solid foundation of truth, and by rearing it with an earnest anticipation of the better things in life, the path of social progress can also be laid out with a fair degree of certainty, and traveled with reasonable safety. The roads to be constructed are many; some have been partly built, others are being surveyed, and others must be projected through what now appear to be inpenetrable jungles. To do so requires knowledge, courage, and character.

The world has waited long for the man with intelligent courage, with skill and vision, who shall pave the way that leads to peaceful co-operation among all men and overcome the discord between nations and the misunderstandings, which, industrially, prevent employers and employees from merging their efforts in the common good. These are the conditions which have obtained in the body politic, despite the efforts to overcome them by men nurtured in older lines of thought. Their removal becomes increasingly imperative.

We cannot question the real desire for peace by those who are to-day striving to settle the economic disruptions that arose out of the World War. We cannot doubt that many employers and employees are earnestly seeking the formula under which both can labor in harmony and satisfaction for their own justifiable ends and the service of society. They all are seeking an adequate solution of the problems; but do they follow the exact methods of hewing to the line of unprejudiced observation and truth by which the enduring engineering structures have been made possible? How fortunate it would be if the formula of broad social service, instead of the rule of narrow selfishness, were used to find the right way out of present bewildering conditions.

The heart of modern societal organization is in its industries. The foundation of our life to-day is essentially economic, and we are recognizing more and more that the world is economically interdependent. The decision of the British Rubber Growers Association to limit the rubber production in the British colonies in Africa, is not a problem of concern merely to the British rubber industry; it may mean a higher price for rubber in the world's markets, which, in turn, may be reflected in higher production costs of rubber articles in the United States, and may even lead subsequently to a reaction on the demand for these products. Therefore, the employment of the rubber worker in the United States, and the welfare of the storekeepers who depend on him for his trade, are inevitably tied up with this decision.

Similarly, on the disputes in the pastoral industry in Australia, as to the living wage that shall be fixed by the Australian Industrial Court for sheep shearers, hinges the price of Australian wool for world export, the quantity of

wool that not only American but English and other woolen mills, will import, the employment of those engaged in the woolen manufacturing industry, and the ultimate price the consumer may have to pay for woolen clothing. In general, an event in some part of the world may readily bring quick repercussions in other parts thousands of miles away, owing to the highly interdependent character of economic organization. Therefore, in all actions taken in the economic field, as in other fields, not only broad vision and thought based on intelligent selfishness, but also on the common welfare, is necessary, but such actions must be grounded on ascertained facts and their proper understanding and evaluation. Facts constitute the solid groundwork on which we can all build with safety.

The training of the engineer should fit him particularly for leadership in industry as well as in public service, because his method of work brings him constantly in contact with reality, which implies facts and human experiences; and on the basis of his knowledge of these facts and experiences and of his scientific method of approach, he should be able to formulate workable and effective rules and plans for the solution of the problems that arise.

Just as it is necessary to depend on him who has a correct knowledge of engineering for the erecting of towering buildings and colossal bridges, so the social and economic structure in which we live must be designed and erected by the application of those rules which have their roots in the mathematical formulas of exact thinking, with due consideration, however, for human variables. Lord Kelvin has so truly pointed out to the engineer, as regards work in his professional field: "When you can measure what you are speaking about and express it in numbers, you know something about it, but when you cannot measure it, when you cannot express it in numbers, your knowledge is of meager and unsatisfactory kind." The engineer, however, as a leader in social progress, must broaden this concept of a mathematical character by applying to it due realization and understanding of the fundamentally emotional relationships of persons as well as of groups, bearing in mind that the psychology of a group often differs materially from that of the members composing it.

The particular contribution of the engineer to the world's work lies in the fact that his is a constant spirit of inquiry, of a pioneering endeavor to bring the experience and exact knowledge of the past to apply to definite situations of the present and to probable exigencies of the future; and in the further fact that his is a spirit of open-mindedness born of improvement as inherent in scientific work. The enhancement of these qualities in the coming engineer is a particular task of the engineering schools; but they must go farther than heretofore in equipping engineers for their work, if it is to be lifted from that of routine to that of constructive leadership. Although it must be the aim to help each engineering student to make his maximum contribution in the common service, the engineering school must cultivate that element in his personality through which he will reach the highest level of usefulness. Without neglecting to teach the necessity for united and correlated action, the instruction must emphasize individuality. It must draw out what is best in each one, searching always for means to promote the good, and eradicating

those elements which would oppose true service. The schools have merely arrived at the gateway of character building, and must now discover paths which lead to trees bearing wholesome fruit.

Fostered by necessity (that great mother of invention), engineering schools have developed according to their various and several opportunities. They seized the lamp of science, but, in order that they might pass it on to shine more radiantly and more fully, they had to erect first the material structures in which to live and labor, and then to search and bring together the store of known knowledge and a body of earnest men to vitalize and make it productive of immediate good and further growth. That they have done their work well is shown by their remarkable development. Due to circumstances inherent in the necessary steadfastness of educational institutions, these schools, from time to time, will lag behind in the march of progress, and it is then that we must take stock to see how they may be brought once more into synchronism with their industrial period and with advanced social thought, in order to assume their rightful position in the great social, political, and industrial progression of the nation and the world.

DISCUSSION

ROBERT FLETCHER,* M. AM. SOC. C. E.—In general, the principles and methods set forth in the papers on Engineering Education, and the suggestions offered for securing a broader and more thorough training at the start, have been followed at the Thayer School of Civil Engineering, in Dartmouth College for about thirty years. Indeed, the beginning was made fifty years ago with a six-year course.

Professor Swain's view of the subject of engineering education is most admirable. The speaker has been fighting for nearly fifty years, as a teacher, against this idea of premature specialization; it is wrong. He has endeavored to make the young men under instruction see the essential unity of the fundamentals that dominate the entire curriculum which the would-be civil engineer should pursue. Mathematics, physics, mechanics, and their immediate applications in the science and practice of engineering, constitute a permanent and indispensable foundation, the breadth and depth of which cannot be diminished. The task is to give a man such a thorough grounding in those fundamental principles, and their applications, that he will not fail to see how they underlie and direct all that comes afterward. The increasing host of modern developments seldom presents anything new in principle. Even the phenomenal achievements of to-day in engineering practice mostly exemplify principles that were learned long ago, sometimes, it is true, expanded and extended as the result of modern research.

F. B. SANBORN,† M. AM. SOC. C. E.—The speaker will discuss only one phase in the new methods of engineering instruction, namely, that which concerns the fundamentals of engineering education. What are the fundamentals? Shall there be one list of fundamentals for an engineering school that is giving a practical education and another for an engineering school that is giving a theoretical education? A variety of lines of progress should be available, and a student should be able to select the school that is best adapted to his needs. One type of student might require for his best advancement that the practical be presented first, whereas another might require exactly the opposite. The courses of many engineering schools embody two years of theory, followed by two years of applied science. The fundamentals in such a course commonly include algebra, geometry, trigonometry, calculus, physics, chemistry, English composition, English literature, history, French, German (or one other language); but if the curriculum is arranged for practical courses to come first, the fundamentals would include field surveying practice, practical laboratory tests in physics and chemistry, mathematical problems associated with their field and laboratory tests, likewise English composition, including expositions and letter-writing based on observations made in studies. Also, the beginnings of courses in finance and economics might be given as fundamentals. The more advanced courses in mathematics, literature, history, and language would be given during the last two years of study.

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The speaker contends that the theoretical-first method need not be applied to all students; in fact, his experience as professor in four different universities and colleges has proved to him that many students advance more satisfactorily by studying the practical demonstration first.

The importance of a new alignment of fundamentals was emphasized a few years ago by Professor Mann in his report to the Carnegie Foundation. He made an exhaustive study of engineering education, in school, among graduates, and among employees of graduates. As a result of this study he emphasized the human element as a feature that should be developed in engineering education. He placed character building ahead of even computations and design. Character building was a new fundamental. It could be instituted in a college curriculum only by displacing some other line of study. What study should be displaced to make room for the study of character building? As Professor Mann's report became digested, the officials of many institutions saw that if a real course of study in the human element were to find a place in the first two years of college study, it must displace fundamentals which must be moved to the later years and perhaps not be classed as fundamentals.

The establishment of courses in character building has progressed considerably since that report was presented. There is still opportunity for wider introduction of these fundamental subjects. The speaker has found it possible to introduce character-building courses in definite forms, not by giving such studies as incidentals and small parts of other courses, but as major parts of definite courses.

Various studies may be included properly under fundamentals. Each institution should be urged to select and to specify the courses of study that it proposes to furnish. Many college catalogues are too sweeping in their claims. It is not practicable for an ordinary college to give a course of study that is well adapted to the students who have a practical bent of mind and also to those who have theoretical inclinations. An institution cannot cover all the fundamentals and all the branches of the Engineering Profession in other than a cursory manner.

It would be better, say, for one institution to furnish instruction in the fundamentals of practical engineering; another to teach the fundamentals of a theoretical education; and a different institution to qualify its graduates primarily for positions that combine business with engineering. Progress is a necessary step in present-day engineering education. These papers, the remarks of other discussors, and Professor Robinson's book "Mind in the Making", all point to the need of changes. It is a sign that indicates growth and activity. As new methods are considered, it is hoped that a new alignment of fundamentals will be emphasized; that there will be different types of engineering schools, some with practical courses of study first, followed by the theoretical; others with the theoretical courses first, followed by the practical; and that each school will state clearly and definitely what type of training it offers, and thus guide students in selecting a training which will be the most advantageous.

The speaker seconds the criticism by Mr. McCrosky.* Such criticism refers specifically to the objection of requiring too much in each course of study.

J. K. FINCH,† ASSOC. M. AM. SOC. C. E.—The papers presented by Professor Scott and Dean Raymond emphasize strongly the fact that engineering instruction should consist largely of broad and thorough training in the fundamentals. In past years, there has been a tendency toward so-called vocational instruction which will produce graduates of high technical proficiency, will make well trained draftsmen and designers, but will not make engineers. Dr. Fletcher has stated that instruction at the Thayer School of Civil Engineering at Dartmouth College has always been based on the plan of thorough training in the fundamentals. The engineering schools of Columbia University were founded in 1864 on this same plan, and the course in each branch of engineering has always included thorough instruction in the sciences, mathematics, physics, chemistry, mineralogy, and geology, as well as in the fundamentals of the other branches of engineering.

In addition, Dean Raymond's paper discloses clearly the growing demand not only for this thorough and fundamental type of engineering training, but also for a broader kind of training which will fit the future engineer to meet the requirements of the rapidly broadening field of engineering effort. The engineer to-day is being called on to meet new problems in new fields where the engineering viewpoint and engineering methods are found to be necessary. Indeed, such opportunities are, in general, more attractive to young men of broad training and high ambition than more strictly technical positions. The paper shows that a careful analysis of conditions in the West argue for a five-year program, allowing time for some of the non-technical subjects such as English, history, and economics, as the best means of meeting this demand. About ten years ago, it was concluded that the proper solution of the problem of a liberal engineering training was to require engineering students at Columbia to take a college course before their engineering studies, as had already been successfully done in the courses of law and medicine. Those in authority do not advocate that kind of a demoralizing country-club college course which Professor Swain denounces in his book, "The Young Man and Civil Engineering", but they do feel that the ultimate solution of the problem of a broad training for the engineer will be to base, as they have already done, his engineering training on a good sound college course. They do not think that this plan can or should be followed in every institution. Columbia which is a great privately endowed institution and which, therefore, is free to do what is felt to be best in advancing the standards of higher education, has met this demand in a manner that is thought to be the best and which will ultimately be so regarded. As an ideal, it is certainly worth working for. As a practical plan, it may be so far in advance of general engineering and public opinion that no institution can afford to make it obligatory.

* See p. 1268.

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What has been the result of this change, which lengthens the period of university study from four to six years? As far as attendance is concerned, the result has been just what was expected. The new plan went into effect at a most unpropitious time, but even with the conditions growing out of the World War, general business depression, etc., the attendance is about one-half the normal capacity and is increasing. It is believed that the ideal of a broadly trained engineering graduate is growing in the United States, the papers presented on this subject prove this, and that, in due time, Columbia University will receive the full support which its program merits. Certain of its requirements have doubtless been excessive and its procedure may have lacked in flexibility but, with modifications in details, those in charge are prepared to carry out this plan which they believe to represent a high ideal in engineering education.

The other result which was also expected, is that the students entering the engineering school are more mature in mind and more thoroughly fixed in purpose than those in the four-year course. Although the engineering course is in no sense a graduate engineering course, it is possible to pass over the elementary points in instruction much more quickly with these men than with the usual engineering student, and to devote more time to discussing matters about which there is room for a difference of opinion, for careful analysis of assumptions and practice, in short, to emphasize the development of an "engineering mind", rather than merely dwelling on facts and standard methods.

There are two other points on which the speaker would like to comment: First, Professor Scott has spoken of the necessity of training men for certain positions. The experience at Columbia is that this cannot be done in a university. Not only must university instruction be broad, fundamental, and thorough, which will crowd out much special training, but graduates often take up branches of engineering other than those in which they majored in college. Mining engineers become civil engineers, for example, and quite recently a Columbia civil engineering graduate was appointed to the presidency of a large company, his two predecessors having been Columbia men, one of whom, however, was a mining engineer and the other an electrical engineer.

The other point which would seem to be of vital interest to the Society, is that if an engineering graduate goes into the broader field of engineering effort, as many are doing, in which engineering furnishes a background rather than the main subject of his efforts, in most cases, he no longer considers himself an engineer. There have been cases, for example, of Columbia graduates having gone into bond investigation work for banking houses, who have either never joined the Society or have given up their membership, perhaps to join a banking association. If such positions offer great opportunities for engineers and if, as seems to be true, it will be the duty of future engineers to meet many problems in other than strictly technical fields, it seems obvious that a determined effort should be made to keep in the Society these men who are carrying engineering methods into new fields which, if not strictly en-

gineering from the technical standpoint, are nevertheless engineering from the standpoint of method, thought, and action. A large part of future engineering will be in this direction, and it is the duty of the Society to claim this field and to keep closely in touch with it. Many technical contributions will be found in the publications of the Society, but little has been done either to interest and to hold engineers who are in these modern fields of engineering work or to advance the application of engineering methods in other than strictly technical lines. This is an opportunity for the Society to increase its interests and activities and to hold in its membership an important group with which it is in a fair way to lose contact.

GEORGE F. SWAIN,* PAST-PRESIDENT, AM. SOC. C. E.—There seems to be an idea—and it is well founded—that something is wrong with education. It seems to the speaker that one trouble is with respect to the fundamental question as to what education is. He has referred to this in the book mentioned by Professor Finch. Ex-President Hadley, of Yale University, although he, perhaps, had not seen the speaker's book, has taken that idea as a text, and has written an interesting article,† entitled, "What Is Education?"

Education, in reality, means a drawing out, an effort to develop the native, inherent, potential possibilities of the student. It has been largely interpreted, of recent years especially, to mean "to put in". The student thinks he is going to school to open his mental mouth and be crammed with information. That idea is wrong, and will never lead to good results. The object should be simply to train the mind of the student by proper subjects of study and proper methods of instruction. He should be taught the fundamentals of science, mathematics, mechanics, and certain branches of engineering, also some history, English, economics, and languages, and should become possessed of a mind trained so that he may be able to turn it to the consideration of any given subject, and bring to bear his knowledge of any or all fundamental subjects that he has acquired. The tendency has been in the other direction. The field of science has increased enormously within the last few years; new subjects have been developed—the automobile, the tank, the aeroplane, radio—but they all depend on the same fundamental principles, most of which were known years ago. Instruction in those specialized branches should not be given in the undergraduate course. Only the fundamentals that have been named should be taught, and the student should be allowed to come back for graduate courses, as has been stated, if he wants to pursue specialized branches further.

The tendency to give information rather than training has gone back into the preparatory schools, and the curricula of those schools are being crowded with subjects which do not belong in them. When the speaker was a boy, it was said that the child was taught the "three R's". He does not know what they are taught now, but he does know that they do not get the "three R's", and that even college graduates do not have them. They have been treated to a

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† *Harper's Magazine*, December, 1922.

superficial taste of many different subjects, generally without learning any one of them thoroughly.

If educators could get back to the fundamental idea that they want to draw out innate power, that they want to train, instead of to put information into the student's head—although, of course, incidentally, they inculcate a great deal of information—that the main idea should be the training in fundamental principles, they would observe great improvement. The speaker knows that college graduates do not get such training to-day.

How far afield we have gone in engineering education is illustrated by the fact that Professor Turneaure's suggestion, which Professor Raymond has quoted, and which is excellent and sound, is put forward as something new. Professor Fletcher has stated that it is what they have been doing at the Thayer School of Civil Engineering, at Dartmouth College, for thirty years, and these very principles have been applied at Harvard University and the Massachusetts Institute of Technology for more than thirty years. There is nothing new in them; they are fundamentals; they are important; and they should be acted on more than they are.

C. M. Spofford,* M. Am. Soc. C. E.—The speaker is heartily in accord with those who have expressed the opinion that the education of the engineer should be devoted primarily to rigid training in fundamentals, rather than to the effort to give the student a mass of information concerning practical details which are continually changing and knowledge of which can be acquired with much greater advantage in practice than in the schools. These fundamentals should include English, economics, the sciences underlying engineering, and the specific theoretical subjects on the application of which his particular profession depends, these latter subjects being accompanied by practical examples sufficient to illustrate their application, and given primarily for that purpose, rather than for information.

Inasmuch as the character, judgment, and vision of the engineer, as well as of other professional men, are of supreme importance, the desirability of developing these qualities should be borne in mind in all his training; and his various courses should have among their most important purposes the development of thoroughness, self-reliance, honesty, and imagination. It is the speaker's belief that the usual subjects in the engineering curriculum are well adapted to the development of such qualities, and that the exact subjects selected are of less importance than the logical arrangement of their subject-matter, the method of teaching, and the personality of the teacher.

Dugald C. Jackson,† M. Am. Soc. C. E.—The papers by Professors Scott and Raymond, and the discussion by Professor Fletcher, might be taken to indicate that the engineering schools had been preaching certain doctrines in regard to education for thirty years, but that these doctrines have received so little acceptance that it is necessary to repeat them. This seems to be unfortunate; but the papers by Messrs. Alexander and Harrington indicate a different situation in regard to the acceptance of the doctrines referred to, and Professor

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Fletcher is to be congratulated that he was one of the early teachers in engineering to emphasize the ideas which the engineering schools now fully support, and which are receiving the endorsement of the Engineering Profession at large. It takes a long time to bring a general recognition of the appropriateness of educational doctrines. The problems are complex and difficult. There are no means of securing a specific or a formula which will determine the education that should be provided for a particular line of endeavor. There are certain threads, however, which experience has proved must be followed.

There has been a vast change in the attitude of the Engineering Profession toward the engineering schools during the thirty-two years in which the speaker has been a professor in such a school. When, as a rather young man actively engaged in engineering affairs, the speaker went to an engineering school as a professor, one of his intimate friends, also actively engaged in engineering affairs but considerably older, referred to the employment as an engineering teacher as "retirement to the cloister". That represented fairly the view in the Engineering Profession at large at that time. The graduate of the engineering school was looked on with suspicion in the Engineering Profession, as a man probably of erratic tendencies, theoretical but not practical, and there was still a discussion of the relation of theory to practice. Since then, all this has been changed. It is now recognized that the engineering schools are not cloistered institutions for a few who perhaps may have some curious ideals, but are the substantial seat of training for the present and future members of a great profession.

It is often said as a pleasantry, that the only profession in which fathers recommend their sons to follow their footsteps is that of engineering. Whether or not this statement is correct, it is a fact that the sons of a great many engineers enter their fathers' profession, and we must see to it that they have the proper education to enter that profession on a level with its present elevation and accomplishments. Also, whether our successors are our sons or the sons of others, we must see to it that the education of our successors is adequate. We must also distinguish carefully between professional education which must be conferred on our successors and is to be provided in the great engineering schools, and vocational education which is needed for a large group of employees in industry, who do not reach the professional elevation. This is a matter of great moment to be considered and reflected on by the National Engineering Societies.

During the past thirty years the medical societies have taken a greater interest in the education of the physician than theretofore; their interest was followed by rather active work in education for the law by the Bar Associations of the country; but it has been only during the last twenty years that the National Engineering Societies have taken an active interest in professional education for engineers, as exemplified in the performance of the engineering schools.

The first joint meeting of a National Engineering Society (the American Institute of Electrical Engineers, in that instance) with the Society for the Promotion of Engineering Education was held in 1903, and at that time there

may have been great doubt as to the interest of the members in such a meeting. However, it proved so satisfactory that a similar joint meeting was held in 1912, with great satisfaction. In 1921, there was a joint session of the American Society of Mechanical Engineers with the Society for the Promotion of Engineering Education.

These joint sessions are very serviceable. They are encouraging to the teachers, and they should be illuminating alike to the teachers and the engineers in practice outside the colleges. It is to be hoped that many more meetings of this kind can be held, as the teachers receive very definite inspiration and encouragement, the engineering schools are guided to a clearer vision, and the Engineering Profession at large gains through the solidarity secured by the interchange of ideas between the teacher and the practitioner.

At one of the early joint sessions, the speaker ventured to suggest that the engineer's education should be founded on the concept that an engineer should be defined as a man competent to conceive, execute, and administer great works, but this was then considered by others as rather an idealistic view. However, there seems to be an agreement that that is what we are now substantially looking for as the premises on which to build professional engineering education. The question arises: How can the situation be improved, and what can the engineering societies do in further aid? Mr. Harrington has made some definite and important suggestions and some concrete and desirable proposals in respect to engineering students. He has suggested that there is more in the way of co-operation that can be accomplished. The foundation is now laid so well that it might be practicable for the boards of government of the several National Engineering Societies to consider this question and establish a joint committee for the interchange of ideas between the National Societies on this important problem. Such a committee would be serviceable in co-operating with the new committee of the Society for the Promotion of Engineering Education, referred to by Professor Scott, and also in co-operating with the important Committee on the Education of Professional Engineers which the National Industrial Conference Board has convened.

The National Engineering Societies might also add their influence to broaden the activities of the Society for the Promotion of Engineering Education. Such support of the effort to co-ordinate more adequately engineering education on sound premises, which Professor Scott recommends so heartily, will go far toward establishing the study of the situation which he recommends, and which is recommended by the new committee of the Society for the Promotion of Engineering Education. It is necessary to demonstrate that the Engineering Profession believes that such a study is desirable for the benefit of the Profession, the industries of the country, and of the social structure of the Nation, but with that demonstration the necessary impetus will be furnished, and the speaker is satisfied that this important study will be secured. It is to be hoped that this joint session may bring the matter to the attention of professional civil engineers so adequately that the Society may join in supporting the effort which may prove of such great moment in advancing the status of the Profession.

THEODORE T. MCCROSKY,* ESQ.—In view of the fact that education is being discussed, it is perhaps fitting that a student should participate to some extent.

In engineering education there is a definite tendency to overload the curriculum, and this re-acts to deaden the initiative of the student. There are so many fundamental courses which must be included, and so many cultural and practical courses which should be included, that the student who does justice to all has little time or energy left to devote to individual work. A student's time is monopolized to such an extent that he cannot pursue the particular subject in which he is especially interested. His initiative, which prompts him to do unrequired work in some branch of his profession, is stifled by the mass of stereotyped work required by the faculty. In order to satisfy his initiative along special lines, in order to live up to his ambitions, he must neglect his assigned work. Consequently, one hears, over and over again, the question: "Do we have to know this?" If this question were asked only by the undergraduate of the type who looks on his university as a country club, it would not justify mentioning, but, on the contrary, it is asked repeatedly by men who really come to college for an education. This attitude is far from praiseworthy, yet, in a sense, it is necessary.

There is no doubt that the faculty knows best what should be taught. Each student is faced with two courses of action—either to neglect in part his curriculum in order to devote himself to original work, done on his own initiative in answer to the call of his ambition, or, to do his curriculum work as it really should be done, and stifle his initiative and the promptings of his ambition. Which is the better engineer—the man with plenty of initiative, but weak in his theory; or the man who knows his theory, but has no initiative, nothing but the ability to do what he is told? Certainly, neither is the ideal engineer.

This is not a new dilemma, but is one which should never be overlooked. Attention has been called to it, because it has not been raised thus far, and surely, no discussion of engineering education would be complete without some mention of it. An engineering student, therefore, appeals to the great educators of the country to keep it always before them, lest the initiative of their students be killed.

T. CHALKLEY HATTON,† M. AM. SOC. C. E.—A subject in which the speaker has been interested for many years, and has discussed before many universities, is the education of young men so that they can express themselves before committees and boards in such a way that they will be understood.

The speaker does not know the number of hours for the study of English required by the average engineering school or college, but he does know that he has had about fifty-five young engineer graduates on his staff, each of whom are required to send in reports weekly—sometimes daily—of his operations. These men are frequently called on to prepare specifications for their particular work, and it is astounding how little they know of the English language and of the legal side of public work and of contracts. The speaker has also presided over many engineering meetings and societies of young men, and has

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called on those whom he knew understood the subject under discussion, and has been embarrassed in finding how poorly they could express themselves. It seems that although fundamentals are wanted—they are necessary in all engineering education—something broader is needed. A man can learn the fundamentals of engineering, and still be graduated a very narrow-minded man. He must know how to express himself in good English. He must know something about politics, that is, he must be in close touch with what the people want, must know how to find out what they want, and how to express himself in discussing their wants with them.

If universities would give more attention to the teaching of English, the speaker thinks that engineers would make greater strides than they are making or ever have made.

J. C. RALSTON,* M. AM. SOC. C. E.—It is with hesitation that the speaker ventures any comment on so complex a subject as engineering education. He has often thought that there should be some orderly and more human method of classifying students before they are enrolled either in the engineering colleges or the universities, or that there should be some form of probationary enrollment based on something more than mere mathematical equipment—something more effective in human classification than that of continuing the present rather loose and inconclusive custom of accepting the applicant who satisfies certain pedantic requirements. Why should an engineering college enroll a pupil whose temperament is anything but that of the engineering mind?

Could not the matriculant be shaken down through a stack of psychological screens, from the coarse mesh of ineptness to the fine 200-mesh of specific qualities, in the same metaphorical manner as the metallurgist classifies the mechanical properties of his field samples before submitting them to the fire for final measure of worth and value? The speaker has seen scores of young men who have been trained in college as far as the colleges and their splendid staffs have been able to train them, but who did not have the "engineering mind." It would be impossible to produce engineers from such material. The Engineering Profession has adopted incomparable refinements in its laboratories and elsewhere in determining qualities and qualifications of the materials of construction. Have the colleges and universities met the concept of this challenge in their selection of human materials? The forces and materials of Nature are the alpha, and the agencies of human nature are the omega of engineering. Do the demands of college enrollments, blinded by the false notion of mere numbers, smother an understanding of the necessity for the selection of proper human materials?

This subject has been visualized before by many earnest and accomplished educators. The speaker is aware that the land-grant colleges may be unable to enforce desirable restrictions, other than loose educational ones. Possibly, even the less hampered universities have endowment or legal restrictions which administrative officers construe with a meticulous nicety; yet the challenge to

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the engineering educators and to the whole profession as to the selection of the engineering mind still stands.

It is granted that there is no super-man who may say that John Doe may not ultimately make a good engineer, but the speaker is constrained to think that the engineering genius in education is not so devoid of intuition, so lean of devices, nor so hindered with traditions, that it cannot use some of the methods or devices chosen by business and industry in selecting employees on whom special training is commercially worth while. In the speaker's judgment, this constitutes a challenge to the profession.

Another outstanding challenge to the broader concept of American engineering education is the preparation of the young man for the wider activities and duties of the future engineer. Technical education is well understood and digested, but the vision of applied ideals in engineering is overlooked or too vaguely understood.

What class of professional men in the United States has shaped the nation's religious ideals and traditions? Manifestly, those of the divinity group.

What professional group has shaped and guided the nation's political ideals and traditions? What group has inspired the best political development, has constructed the fabric of the nation's highest political and legal aspirations, and made of them a national tradition? It has been the legal group.

Again, what professional group has shaped or is shaping the nation's industrial ideals? None. Industry in America, as far as it has gone, has accomplished superlative scientific and mechanical developments. Our industrial ideals are compounded only of cold material accomplishment. This, however, is only one-half the nation's best aspirations and ideals. The other half, the spiritual or human half, has been left untouched, except in so far as it has been poisoned by the virus of class-consciousness, super-unionism, and grudging service, directed by scolding, whining, and fault-finding groups of doddering agitators. These are the groups and types that are misshaping our real fundamental industrial ideals. The breath of constructive and patriotic genius has been smothered because engineering vision has been microscopic; the view has always been a "close up". The broad perspective has been lost.

What professional group, therefore, is qualified by training, education, and vision to direct and inspire the nation's industrial ideals and shape them into a great national tradition as the other two major national traditions have been shaped? What group has been remiss and has failed in the discharge of a plainly obvious duty, or has failed to prepare itself for its broader activities in its own special field? Conclusively, the engineers.

This, then, is the other challenge—the challenge to leadership in industry, to the Engineering Profession, to the teachers of engineering. Let the challenge be met so that it may no longer be said that the spiritual vision of the Engineering Profession is charged with the exuberance of promise and the sterility of performance.

MILO S. KETCHUM,* M. AM. SOC. C. E.—For some time there has been an attempt to select men for the Profession of Engineering, and many have

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thought that it is possible to determine early in a man's career whether or not he will later show proficiency in engineering work. Since returning to the University of Illinois, the speaker has been strongly impressed with the records of two men whom he knew in their student days. These men were students in mining engineering, and were about the two most unpromising candidates for the Profession of Engineering that could be imagined. Based on present standards for rating men, these two were lacking in practically every essential that would appear to qualify them for the Profession of Engineering. One of these men was graduated, and the other "quituated". The former, at a later date, received a Ph. D. degree from a prominent Eastern educational institution, and is now one of the leading mining engineers in America. The other has also made an excellent record in mining engineering. Students of the caliber of these two men would not at present be permitted to remain in any self-respecting institution.

Early in his teaching career, the speaker thought it possible to determine whether or not a young man would make a success as an engineer; but, during twenty-five years of experience, he has been unable to find any method of selecting young men for the Profession of Engineering. There are so many elements that go to make up a successful engineer that no method of rating has been discovered, which will be of any material assistance in determining whether young men should enter that profession.

Referring to the matter of the curricula of engineering colleges, the speaker has felt that perhaps we are losing our heads in our desire to make a man out of a boy—to get accomplishments at 21 that we should only expect at 40. It is difficult for a young man in college to be both broad and deep. Some months ago in talking with an alumni committee that was trying to determine how a certain course in engineering should be given—that this should be put in and that should be put in—the speaker finally said to one of the men:

"If all the courses that your committee is suggesting were placed in the curriculum, the student graduating would be very much like the Platte River. The Platte River is a mile wide and two inches deep; it goes wandering all over the plains; it sticks itself into everybody's business; it drops things here and there; and it accomplishes nothing. It is just a pestiferous nuisance. If you were to confine the waters of the Platte River within a narrow channel, it would be a self-respecting and useful stream; its velocity would be increased, and it would carry away the sediment in place of depositing it along the way."

In a professional way, most of us are inclined to be like the Platte River. We cannot be both broad and deep, and the speaker has come to the conclusion that we should give the most attention to making the engineering student deep, paying considerable attention to his ambitions and aspirations, giving him enough breadth so that he will finally become a useful citizen and be able to accomplish something in his chosen profession; and in that way we shall be doing the most we can hope to do.

The speaker has recently been much impressed with the lack of training of many men who have been engaged in research work. The records of all societies are cluttered with the discussions of many subjects and much

experimental work, not only useless, but vicious. Tests that have been well thought out, have been carried on, in many cases, by men without the necessary underlying theoretical training to realize what was intended to be accomplished, the result being that their data have been worse than useless. It is important that research work should not only be outlined by competent men, but that the details of the investigation should be carried out by men thoroughly well-grounded in practical and theoretical applications.

In addition to having been an engineering teacher, the speaker has had considerable experience in practical engineering in contracting work, and in engineering administration. The men in the speaker's employ who have given the best results have been those who made the best records in college and had a thorough theoretical training. These men were able to take up the business and practical side of engineering much more quickly and efficiently than those who had had superficial training along these lines. The broadening and humanizing value of any subject is largely in the teacher. After all, it is the great teacher—the man behind the gun—that is going to count; and it is not so important whether he teaches one subject or another if he puts the punch into his work, if he teaches the young fellow to be awake, to "get into the game", and to keep on growing.

W. E. WICKENDEN,* ESQ.—The speaker has been much impressed by the fact that a unity of objectives is sought, although through a diversity of agencies. The possibility of the Society for the Promotion of Engineering Education, as a liaison body between the great groups of engineers in the National Societies, is an idea that is growing in thought and attention.

The speaker will say little more than to second heartily the suggestion that possibly it is time for each of the great National Societies to appoint a representative to confer with the committee that is already established by the Society for the Promotion of Engineering Education and the National Industrial Conference Board. The speaker has a strongly growing feeling that we are face to face with the question as to whether the lines of cleavage now found in the great engineering fraternity may not have possibly outlived a great deal of their usefulness. Did they not have their roots in an historic situation? Did they not originate in the evolutionary conditions by which each branch of engineering practice has differentiated itself from the parent stock? Are not the activities of engineers, especially those who share in the great industrial enterprises of the country to-day, subdivided along functional lines rather than on lines of historical evolution of the great engineering societies? If we have this question before us, certainly a body which is interested in the subject of engineering education has a great purpose to serve as a liaison body by which the aims, the ideals, the objectives of the great National Engineering Societies may be brought out of their present state of diversity and into a state of unity, as we approach our common objective of increasing the thoroughness of fundamental processes of education by which men are made ready to perform their tasks in the world and carry on the processes of their own education after they have been graduated from college.

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SYDNEY WILMOT,* ASSOC. M. AM. SOC. C. E.—The plan proposed by Mr. Harrington, and amplified by Professor Scott, seems to involve joint action by the Engineering Societies in pursuit of student chapter activities. This is one of the most important phases of our professional contact with engineering schools, and deserves the heartiest sentiment in its support.

There need be no argument as to the mutual advantages of student membership to the student and to the societies. In certain cases, however, when confined to a particular society, even such a simple relationship is almost impossible. Take, for example, a case of personal observation and interest (Brown University), in which the course of study involves three years of unified work by all students, and a branching out or specialization in the fourth. Thus, the student need not decide definitely on his own peculiar trend of study until his senior year. In fact, he would usually hesitate to align himself with any one particular undergraduate engineering chapter. Likewise, a similar inhibition would operate in small schools where the total of students in several classes might not provide the necessary minimum number of men interested.

The speaker has had almost a dozen senior students under his instruction, who might properly in the future become members of the American Society of Civil Engineers, and who would gladly join in some kind of co-operative effort with the Society as student members, if their number warranted such a step.

Obviously, in this case, the present plan is not advantageous for the needs of the situation. Only some kind of joint organization, as proposed among the several societies, will overcome the difficulty. Under this arrangement, all men in the engineering courses, whatever their year or their aim in engineering work, would feel that they were eligible. With such support, the organization might reasonably anticipate success.

At least one other National Society is considering the advisability (if it has not already taken action) of permitting the affiliation of any engineering student or group of students. Joint action, it would seem, is a far better solution.

The sentiment in favor of such a plan seems to justify it, and it is to be hoped that the Society may see fit to advocate it, as it will be an effort of potential value to the Society and of considerable possibilities to the smaller schools.

H. R. BUCK,† M. AM. SOC. C. E.—Mr. Harrington has reported a condition among some of the student branches of the Engineering Societies that is discouraging and may become more so; but he has also presented an interesting picture of an honorary engineering society which would draw together the separate student branches in each of the colleges and technical schools, and which would serve as a common feeder to the four Founder Societies. Is it not practical to ask, what engineers are going to do about it?

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Will not individuals take advantage of the opportunity and write to the Board of Direction, asking that a committee be appointed to try to obtain joint action with the other Founder Societies in improving the work of and the conditions surrounding the several student branches?

A. B. McDaniel,* M. Am. Soc. C. E. (by letter).—There has been considerable discussion in recent years among engineers and educators as to the curricula for the training of engineers. All kinds of combinations of general and technical subjects have been proposed in courses of four to eight years in extent. Many leading educators have strongly urged the modification of the present engineering curricula so as to reduce the number of highly specialized subjects and to provide opportunity for a broader and more fundamental training. Some engineers have recommended that such subjects as business administration, finance, accounting, and public speaking be taught in engineering schools in order to furnish a training in the business phases of the engineer's work. Whatever the plan proposed, or remedy suggested, it is evident that there is a need for a careful and thorough study of existing conditions. Professor Scott has presented the project of the Society for the Promotion of Engineering Education. The purpose of the writer's discussion is to present briefly another project concerned with the study of engineering education.

Experience in army training has shown that the first requirement for effective work is the preparation of detailed statements of exactly what the men must be trained to do. By thus clearly stating the objective for training, it is possible to produce courses of study which are much more effective than those produced under the old system in which specifications are not prepared. Industry has recently recognized the need for such occupational specifications, and many organizations are making the necessary job analyses.

Objection has been made that it is impossible to state the things an engineer must be able to do, and those he must know. Army experience has shown otherwise, however, and has demonstrated the need and value of descriptive statements of the qualifications required for various engineering positions. To meet this need, the American Association of Engineers has appointed a committee of engineers and educators, of which the writer is Chairman, to make a study of the classification of engineering positions. The Committee is confining its attention at first to the highway field, and is securing data from about 3 000 highway engineers in the service of the U. S. Bureau of Public Roads, and six State Highway Commissions throughout the United States. Specifications for the various positions involved will be prepared from the data received. These specifications will be of service to employment services, personnel departments, civil service commissions, and similar agencies in the selection, assignment, and promotion of engineers, and will furnish information to educational institutions for the preparation of engineering curricula.

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R. L. SACKETT.* M. AM. SOC. C. E. (by letter).—There are two factors which cannot be overlooked when the subject of engineering education is treated fairly.

First the average engineering student lacks experience and probably possesses less than that held by those who were graduated ten or twenty years ago. Furthermore, much of the education of the engineer will necessarily follow his graduation, even if five- and six-year courses are adopted. The great school of experience is a finishing college, and nothing can ever supplant it. This is no excuse for ineffective educational methods, but is only a fact which cannot be ignored in judging the efficiency of engineering schools and colleges.

Second, the importance which a student attaches to, say English, by way of illustration, is dependent on his home training, his association with engineers, if any, and the English used by his engineering instructors in regular class work. The importance that other students in his environment attach to English is determined in a similar way. Instructors are employed who were graduated some years ago, but had their experience under engineers who did not exemplify good English. These instructors are sometimes deficient in their working knowledge of good expression, either oral or written. This is also true of instructors in other than engineering subjects. No discussion of engineering education is complete if it does not realize that the salaries paid to engineering instructors are inadequate and do not encourage a prospective engineering teacher to take a four-year liberal arts course and then a technical training in addition. More mature men are needed as teachers of freshmen, not only in teaching English, but as instructors in all subjects. Although habits of language have already become more or less fixed, they are more flexible during the freshman year than later. Better salaries would make it possible to employ men of broader training, larger culture, wider experience, who not only would teach the student, but would also exemplify good English and careful analysis, and unconsciously deepen the student's appreciation of other important factors in his early education.

HOWARD CONSTABLE,† M. AM. SOC. C. E.—Aspiration for a better method of education is evidently abroad among engineers. The engineer starts with mathematics, struggles toward Truth, and neglects a great deal of the broad meaning of Art; the art of understanding and dealing with his fellow man, the art of recreation and inspiration, and, in short, the art of living a rounded out life. How often one finds him ignorant in some fundamental; unfamiliar with parliamentary law; unable to express himself clearly; or awkward in social relations.

The speaker would ask, "What are you going to do about it?". Men, money, methods, and materials, is the order of importance in present-day problems. The human part is generally the most discouraging. Of the men who go into business, 95% fail largely because of inefficient minds, which indicates a rather low standard for the intelligence of the nation. No wonder there is a growing demand for a revision of the educational system.

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The speaker has often advised men to try for jury duty. In that work, one may learn much of human nature, that what is written is remembered and what is spoken is forgotten; that there is no gratitude in compromises, yet compromise is the general rule; that every transaction may lead to a lawsuit.

Another matter in education is the question of whether the ability and wisdom of men in active practice is utilized as much as might be, for the benefit of the student and the public? In this age, interest in such things is broader than just technical societies. Should not great engineers be more called on by the colleges and the public?

W. G. PRICE,* M. AM. SOC. C. E. (by letter).—Efficient engineering education is a subject that apparently is least understood by so-called educators. The colleges are not efficient educational institutions, and neither can one get a well-rounded education in the practical school of life outside of college. No schools have been provided for educating all the brain faculties of the growing life. It has been said that college graduates know everything, and have no sense. It may also be said that those who have been limited to gaining sense in the practical school of life have otherwise not much education.

A child is endowed by nature with undeveloped perceptive, initiative, creative, and executive faculties, also with ambition, confidence, memory, and love for his fellows. All these faculties must be developed in order to gain a complete education. The cultivation must be intensive and continuous during all the formative period of life. No one of these faculties can be given normal strength after twenty-one years of even partial idleness. In order to develop the greatest muscle power, one must strain it each day to its limit of strength during the growing life, and the same exercise is necessary to put each faculty of the mind in the condition of maximum efficiency.

Antioch College is combining the acquiring of knowledge with creative and executive work, but is beginning too late in the life of the student. The initiative, creative, executive, and ambitious faculties, all of which are comprised in "doing without being told how to do", must be cultivated in childhood and as long as life lasts, in order to make the life efficient. Vocational work in schools is not accomplishing much, as most of the work is copying what others have done, and, therefore, is memorizing and not initiative. An engineer should be a resourceful leader.

The memorizing of useless information weakens the mind. Much of school and college memorizing is useless in engineering. The acquiring of much general knowledge by the student is necessary but mind power is developed by doing, and not by learning, what others have done. Usually, the faculty of wanting to do is the first mind force manifested by the child. Schools should cultivate and direct that faculty. Colleges frequently graduate men who have but little ability in engineering or business. Where nothing has been planted, the harvest will be nothing.

CHARLES F. SCOTT,† ESQ.—With the general purpose of indicating the evolution which has taken place among engineers, along the lines indicated by

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Mr. Wickenden, the speaker will discuss student branches, and may indulge in some personal reminiscences.

Twenty years ago, the speaker was President-elect of the American Institute of Electrical Engineers, and made it a point to ask many of his colleagues what the Institute should do, and what its function was in the electrical engineering field. One of the replies was this: "I do not know what the electrical industry is going to be in the future, but in the past it has been doubling every five years."

The speaker knew that the progress of the past had been rapid, but, not having thought ahead, was startled by the idea that in a short five years we were likely to have doubled what then seemed a very wide use of electric service. Now, if in five years the volume of electrical industry was to be doubled, and in ten years quadrupled, where were the men coming from to handle the great task? From the schools? Then, to be an instrument in the development of electrical engineering, the National Institute should direct its attention toward the schools, in order that the students might be able more quickly and more effectively to take up the burdens of engineering work after graduation. That thought was followed by correspondence with several engineering professors—this was while the speaker was in industry, before becoming a teacher—and, as the outcome of the replies and a recommendation to the board of directors, student branches were authorized and, within the next few months, were formed in a number of schools. Others followed, until now there are sixty or seventy. Two or three years later, the mechanical students in one of the colleges, desiring to follow the example of the electrical students, formed a branch and made application to the American Society of Mechanical Engineers for recognition. A few years later, the mining engineers formed branches, and two or three years ago the civil engineers started branches.

The idea of a common co-ordination between engineering societies with relation to student activities is so far in advance of the ideas of twenty years ago, that it has come to the speaker with somewhat of a shock; but, in the excellent arguments that have been presented, there are many reasons in favor of engineering students joining together in common effort in student activities. Whether this had better be one group, with subdivisions, or several divisions co-ordinated or federated into a general group in a school, or whether all the students in common, irrespective of the courses they may be taking, had better belong to one general engineering society, the speaker is not yet ready to say; but the coming together of the engineering societies in one building, making it a center of common engineering representation to the students of the country, would be a fine example of co-ordination and one of the best things that the engineering societies could do in influencing the students in the colleges.

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THE RESEARCH ACTIVITIES OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

By Arthur N. Talbot,* Past-President, Am. Soc. C. E.

Engineering art is based on a variety of things, ingenuity, experience, judgment, rules and regulations (even rule-of-thumb), precedent, vision, the laws of Nature known as physics and chemistry, and the special applications and adaptations of the laws known as engineering science. In the early days of engineering, ingenuity, experience, and judgment, were the dominating influences, and engineering science played a minor rôle. As time went on and experience grew, rules and regulations became more important. The increasing magnitude and difficulties of engineering projects, the need for greater permanency, economy, and safety, and the varying and diversified nature of engineering work, brought forward the importance of basing engineering art more fully on engineering science. The physical sciences grew, giving greater opportunities to the engineer; and, conversely, the wonderful development of engineering stimulated the development of both pure and applied science. Altogether, the growth and development of engineering in the past generation has been wonderful. It will not be overlooked by any one that engineering must continue to need ingenuity, experience, and judgment, and rules, precedent, and vision, but more and more, as time goes on, will it be dependent on science and that accumulation of applied science which may be grouped under the term, engineering science.

The growth of a science is not spontaneous; the fruit of engineering science is dependent on well-directed effort, the expenditure of time, energy, and money, and this fact should always be borne in mind. The instrumentalities that are available for the investigation of engineering topics are much the same as those which are useful in the development of other lines of applied science. The industries have their laboratories, some for private and, perhaps, selfish ends, some which give out results generously to the profession and the public, or even co-operate openly with public agencies. The railroads have done much in engineering research. Private laboratories are contributing to the fund of knowledge. The Federal and State bureaus and departments are continuing to give valuable contributions to engineering knowledge, as instanced by the work of the U. S. Bureau of Standards in cement, concrete, and structural materials, that of the U. S. Bureau of Public Roads in soils, road materials, and road construction, and that of some of the States in road construction and maintenance. The laboratories of engineering schools all over the United States are conducting considerable engineering investigation in a promising way, some through organizations called engineering experiment stations, others in less formal manner. A recent report of the organization of Landgrant Colleges enumerated more than one hundred researches now in progress in the engineering departments of those colleges, nearly all of which have some

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application to civil engineering operations. Many schools that are not active in engineering experimentation, would be pleased to have the opportunity to utilize their laboratory equipment for some productive work. It is evident that, as the years go on, the engineering college will take an increasingly active part in research. Finally, the engineering society may be named as one of the instrumentalities from which development of engineering science should be expected.

To quote from a former address,* a National Engineering Society should be (among other things), "a collector and disseminator of engineering knowledge; a forum for the discussion of engineering questions, simple and complex, small and large; a stimulator of research and of the progress of engineering science; a developer of engineering methods and practice, * * *". Is not being "a stimulator of research and of the progress of engineering science" an important function of an engineering society which has for its object the advancement of the science of engineering in the several branches? Is it not within the province of the engineering society to promote the conduct of research, to assist and co-operate in its extension, and even to participate in its activities? Its members know the needs of the profession, can forecast to a considerable degree the trend of engineering development, and should be able to outline, direct, study, and interpret investigational work; and such work may be expected to furnish contributions to engineering knowledge, that will add greatly to the advancement of engineering art.

Many of the engineering societies have participated freely in research activities. The American Society of Mechanical Engineers has had a Committee on Research for years and has made important contributions to engineering science. Its monthly journal has a well-conducted department which digests and reports the available results of research applicable to mechanical engineering. The American Society for Testing Materials has long been a most important instrumentality for organizing, stimulating, and reporting engineering research; it has secured the hearty co-operation of both industries and institutions, and its volumes are replete with valuable contributions. The American Railway Engineering Association has likewise contributed much to engineering science, through laboratory, field, and office investigation. Reference should be made to another instrumentality, closely allied to the four Founder Societies, Engineering Foundation, largely the conception of Ambrose Swasey, Hon. M. Am. Soc. C. E., who generously started its endowment "for the furtherance of engineering, for the advancement of the profession and the good of mankind". Engineering Foundation has worked to encourage, direct, and develop research agencies and activities and, at the same time, give such financial aid to research work as has been found practicable. The Division of Engineering of the National Research Council, an organization formed during the World War and now operated for the purpose of securing the co-operation of engineering agencies in which research facilities are available, has continued to be very active. Its Advisory Board for Highway Research has been conducting important engineering researches. The work of all these organizations may be classed under engineering society activities.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 416.

The American Society of Civil Engineers has a large amount of engineering research to its credit, although it has not had an aggressive research policy. The *Transactions* contain a large number of individual research contributions, many of them of the highest value. One need only call attention to the paper on "Experiments Relating to Hydraulics of Fire Streams",* by John R. Freeman, Past-President, Am. Soc. C. E., that on "Experiments at Detroit, Mich., on the Effect of Curvature upon the Flow of Water in Pipes",† by Gardner S. Williams, Clarence W. Hubbell, and George H. Fenkell, Members, Am. Soc. C. E., and that on "The Economical Design of Reinforced Concrete Floor Systems for Fire-Resisting Structures",‡ by John S. Sewell, M. Am. Soc. C. E., to illustrate the quality of these personal contributions. The rules of the Norman Medal provide that "the medal shall be awarded to a paper which shall be judged worthy of special commendation for its merit as a contribution to engineering science", and the rules of the J. James R. Croes Medal and the James Laurie Prize recognize the usefulness of contributions to engineering science. The lists of awards are full of papers giving reports of scientific researches, both experimental and analytical, and the *Transactions* contain very many more that were not recognized in this way.

Many of the special committees which have been appointed throughout the life of the Society have reported research information of value. The report§ of the Special Committee on Uniform Tests of Cement gives experimental information which was of great interest and helped to establish standards which have proved to be very useful to the Engineering Profession. Other committees, like the Joint Committee on Concrete and Reinforced Concrete‖ reporting in 1916, conducted tests on which to base parts of its recommendations. The Special Committee on Steel Columns and Struts¶ carried on an elaborate set of tests of columns and reported results of much value. In later times, the Special Committee to Report on the Bearing Value of Soils for Foundations, etc., which is still continuing its work, has carried on considerable experimental work. The Special Committee to Report on Stresses in Railroad Track, formed in 1914, has conducted experimental work on a large scale and has presented its third formal progress report.** The three reports contain a great deal of information on the action of the track structure, which is considered very valuable by railroad men. It should be added that this Committee has been co-operating with the American Railway Engineering Association and the American Railway Association.

Altogether, it is evident that the papers of the Society and the activities of its meetings and committees include contributions to engineering science, which have exercised an important influence in the development of engineering art, and that the Society has been an effective instrumentality in the extension and development of engineering science.

* *Transactions*, Am. Soc. C. E., Vol. XXI (1889), p. 303.
† *Transactions*, Am. Soc. C. E., Vol. XLVII (1902), p. 1.
‡ *Transactions*, Am. Soc. C. E., Vol. LVI (1906), p. 252.
§ *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 665.
‖ *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1101.
¶ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1583.
** See p. 907.

For the purpose of formulating and conducting a more active and systematic research policy, the Board of Direction has created a Committee on Research. The minutes* of the meetings of the Board of Direction of January 18 and March 8, 1921, give the following three actions in creating this Committee and defining the scope of its work, according to recommendations made by a special committee of the Board:

"That a Committee on Research be appointed which shall consider and deal with matters of research which may be taken up by the Society and which shall make recommendations concerning research that in its judgment should be taken up by the Society—whether the research work be undertaken by the Society, or be done in co-operation with other societies and committees, or be promoted or stimulated by the Society."

* * * * * * *

"That the Committee on Research and the representatives of the Society on the Advisory Committee be not restricted to members of the Board of Direction and that the element of reasonable continuity of membership be accepted as desirable."

* * * * * * *

"The duties of the Committee on Research (to consist of seven or nine members) shall be to organize, stimulate, and supervise the research work of the Society as conducted by its Committees or through co-operation with societies and individuals, in accordance with the direction and regulations of the Board of Direction."

It will be seen that the Board of Direction had in contemplation plans for co-operation and the stimulation of activities in engineering research.

At the meeting of the Board of Direction of January 18, 1921, the same special committee reported on the letter† of Dr. C. A. Adams, Chairman of the Division of Engineering of the National Research Council, inviting the Society to become the Sponsor Society in research work in Civil Engineering, and to appoint representatives on the Advisory Committee on Civil Engineering. The following extracts* will indicate the nature of the reports:

"The Committee has given consideration to the project and to possible relations to research activities of the Society. It feels that research work in engineering should be promoted by the Society in such ways and to such extent as seem practicable for the Society to undertake. The proposal to co-operate with the National Research Council seems a promising method of stimulating and advancing engineering research. An acceptance of the invitation, however, should not be permitted to involve interference with the right or duty of the Society to advance or carry on research work on its own account or in co-operation with other societies or bodies. The Committee believes that the Society should maintain a cordial interest in stimulating and advancing the progress of engineering science and that it has a duty toward this side of engineering. With a view of safeguarding and stimulating this function of the Society and at the same time co-operating with the National Research Council, the Committee proposes the following:

"That the Board of Direction accept the invitation of the Division of Engineering of the National Research Council to become the Sponsor Society in research work in civil engineering and to appoint representatives on the

* *Proceedings*, Am. Soc. C. E., February, 1921, pp. 168–169, and April, 1921, p. 384.
† *Proceedings*, Am. Soc. C. E., December, 1920, p. 917.

Advisory Committee on Civil Engineering of the National Research Council and to accept other responsibilities connected therewith."

* * * * * * *

"The duties of the representatives of the Society on the Advisory Committee on Civil Engineering of the Division of Engineering of the National Research Council shall be to represent the Society in its responsibilities therein as Sponsor Society in research work in civil engineering, in accordance with the directions and regulations of the Board of Direction. The representatives of this Society on this Advisory Committee shall be not more than fifteen, and shall include the Committee on Research of the Society."

This Advisory Committee was appointed subsequently, and is now called the Advisory Board on Civil Engineering Research.

The Committee on Research was organized on January 18, 1922, with the speaker as Chairman and F. E. Schmitt, M. Am. Soc. C. E., as Secretary. In the following months, discussion was had through correspondence on the division of the field and on what work should be undertaken first. Further discussion was had and action was taken at the meetings of the Committee at Dayton, Ohio, on April 4, and 5, 1922, and recommendation was made to the Board of Direction for the creation of research committees, as follows: Stresses in Structural Steel; Irrigation Hydraulics; Hydraulics Phenomena; Impact in Highway Bridges; and Flood Protection Data.

The Board of Direction at its meeting of April 4, 1922, approved these recommendations and the committees were later appointed and have been actively at work with the exception of the Special Committee on Hydraulics Phenomena which has not yet been appointed.* Progress reports were made by these Committees at the Annual Meeting on January 17, 1923. It may be of interest to note that the Special Committee on Stresses in Structural Steel was instructed to limit its activities as follows:

"This Committee should be instructed to limit its activities at the present time to an investigation of the properties of commercial structural steel (both mill and stock material) as bearing on the problem of what working stresses should be allowed for steel building construction, and to report to the Board of Direction on the feasibility of conducting extensive measurements of stress distribution in structural frame work other than bridges."

At the meeting of the Board of Direction on January 16, 1923, the recommendation of the Committee on Research for the appointment of a Committee on Reinforced Concrete Arches and a Committee on Steel Column Research was adopted, and it is expected that these committees will be appointed in the near future.

The Committee on Research has discussed the needs for research work in the various fields of civil engineering. The number and variety of the topics that call for investigation will convince any one of the richness of the prospect and the wealth of opportunity for service. As new Technical Divisions of the Society are organized, new demands will appear. Sanitary engineering, power, railways, highways, structural steel, concrete, engineering materials, and many other fields, all have their problems. The difficulty lies not in finding the problems, but in selecting those that are the most pressing, the easiest of

* This Committee was appointed on May 17, 1923.

solution, or have the most willing workers. Many topics have been suggested. It has seemed the wise policy not to extend committee work too rapidly, but rather to support and assist the work of the committees now appointed, and to constitute others only as fast as conditions seem to warrant. It is clear to the Committee on Research, however, that there are many fields and a large number of topics that should be investigated by the Society, both by itself and in co-operation with other organizations. It is felt that as time goes on the Society should actively develop the policy of stimulating research in all the branches of civil engineering and of contributing to engineering science in every reasonable way.

The securing of funds to support research may seem a troublesome question. The increasing "high cost of living" has made the income of the Society scarcely equal to its needs, with little chance for expansion in service. In many problems, however, the necessary expenditures will be small. In other cases, after the problem has been outlined, engineers will willingly aid in carrying out the work. In still others, governmental agencies or industrial organizations will be found willing to co-operate. In still others, sums may be raised by subscription from corporations and individuals. The contributions for the work of the Joint Committee on Stresses in Railroad Track may encourage the Society in bringing other research problems to the attention of those who may appreciate the value of a thorough investigation. Engineering Foundation will doubtless contribute as freely as its resources and its other demands will permit. It is to be hoped that its funds will be greatly augmented by new and princely gifts. The Society itself may be expected to contribute in various ways. Its practice now is to pay mileage for attendance on committee meetings. It would seem proper, however, to do more than this—to contribute something to some classes of research that may not appeal to the outside contributor, to do a little, too, in other ways, as an earnest of its interest and faith in the value of such work. This help may not be given at once, but it would seem that the Society should shape its expenditures so as to set aside for the use of research work, as time goes on, a moderate sum for this particular function of the engineering society.

The conduct of committee research work itself, however, will involve time and labor, in conducting the tests, in studying and digesting the data, and in interpreting the results and presenting the findings in an acceptable way, even tedious, dreary, fatiguing hours and days. The men who undertake the work will do it cheerfully, as becomes those who aspire to help in extending the bounds of knowledge, and will find compensation therefor in the satisfaction that they have thereby done some small part in contributing to the advancement of the Profession of Engineering. They should know that the membership of the Society is in a receptive mood and will commend their contribution to engineering science.

In conclusion, it may be well to state that the Committee on Research will welcome suggestions and constructive proposals. It asks for assistance in every way. May not the Society gain greatly by the co-operation of many in these lines of endeavor?

ENGINEERING FOUNDATION: DIVISION OF ENGINEERING, OF NATIONAL RESEARCH COUNCIL: THEIR ORIGIN, WORK, PLANS, NEEDS

By Alfred D. Flinn,* M. Am. Soc. C. E.

In 1914, the spirit of progress and service among engineers found another expression through the farsight and generosity of Ambrose Swasey, Hon. M. Am. Soc. C. E., in his gift of $200 000 to United Engineering Society to be the nucleus of a great endowment "for the furtherance of research in science and in engineering, or for the advancement in any other manner of the profession of engineering and the good of mankind." With this gift as a corner-stone, there is being erected under the auspices of the American Society of Civil Engineers, the American Institute of Mining and Metallurgical Engineers, the American Society of Mechanical Engineers, and the American Institute of Electrical Engineers, an institution known as Engineering Foundation, devoted primarily to conducting and furthering researches and investigations in applications of science to engineering and its related industries. Subsequently, Mr. Swasey increased his gifts to a total of $500 000, yielding an annual income of $26 000.

United Engineering Society is the legal owner of the principal of Engineering Foundation's Endowment Fund. This Society is in effect a board of trustees incorporated under New York State laws "to advance the engineering arts and sciences in all their branches, and to maintain a free public engineering library", with power and authority "to receive, conserve, invest, and reinvest, hold in trust, and administer such moneys or other property as it may receive from any source for its own purposes or for any other endowment or activity within the scope of its charter." This Society has twelve members, three chosen by each Founder Society. It has now two departments: The Engineering Societies Library and Engineering Foundation. Engineering Council was a third department from April, 1917, to December, 1920. United Engineering Society owns and administers for the Founder Societies, Engineering Societies Building and Library and several permanent funds.

The legal status of Engineering Foundation Board is that of a committee of United Engineering Society. The Foundation has full discretion in the expenditure of the income from the Endowment Fund and such other moneys as may be given to it, or to United Engineering Society for it, for research and other projects. The Foundation has sixteen members, elected by United Engineering Society—one trustee representing each Founder Society, two members from each Founder Society nominated by its governing body, who are not trustees, three members at large, and the President of United Engineering Society *ex officio*. Its officers are Charles F. Rand, Chairman, Edward Dean Adams, F. Am. Soc. C. E., and Frank B. Jewett, Vice-Chairmen, with two other members, George H. Pegram, Past-President, Am. Soc. C. E., and H. Hobart Porter, M. Am. Soc. C. E., constituting its Executive Committee.

* Secy., United Eng. Soc., and Director, Engineering Foundation, New York City.

The Treasurer and Assistant Treasurer and the Counsel of United Engineering Society *ex officio* function for Engineering Foundation. The Director of the Foundation is also its Secretary. The Foundation has offices in the Engineering Societies Building, which are provided by United Engineering Society.

Engineering Foundation, therefore, is not an outside, independent institution, but, like the Library, is an integral part of the organism of the Founder Societies and of each one individually. It was created by them on the basis of Mr. Swasey's gifts, should have their continuing support, and should be utilized by them. It is their joint instrumentality for research and investigation, especially for such projects as involve more than one branch of engineering and science as do most researches and investigations of interest to engineers. As its resources are increased, its service can be greatly extended.

In 1916, National Research Council was created at the request of the President of the United States, by joint action of the National Academy of Sciences, National Engineering Societies, and Scientific Associations. It became, in 1917, the Division of Science and Technology of the Council of National Defense. Necessarily, it had a Committee on Engineering, with a number of sub-committees, which did notable service during the World War. National Research Council is based on the charter of the National Academy of Sciences given by Act of Congress in 1863 and signed by President Lincoln. Its legal status is that of a committee of the Academy. It was perpetuated by the Executive Order issued by President Wilson on May 11, 1918, which, among its provisions, directs the heads of Governmental departments to co-operate in every way required. Therefore, although the Council is not a Governmental bureau, it has intimate relations with the Federal Government. Except for war purposes, the Government has provided no funds for its support. All its financial resources come from private institutions and individuals, except to the extent that Governmental bureaus participate in research projects of interest to them. Largest among the gifts are $5 000 000 from the Carnegie Corporation of New York for a building and endowment; $500 000 from the Rockefeller Foundation for research fellowships in physics and chemistry; and $500 000 from the Rockefeller Foundation and the General Education Board equally for research fellowships in the medical sciences. About $1 330 000 are being expended on a permanent building near the Lincoln Memorial, in Washington, D. C., to be ready for occupancy in the fall of 1923.

Engineering Foundation had an important part in the establishment of National Research Council. When organized, the Council had neither funds nor office. The Foundation at once placed at its disposal offices in Engineering Societies Building and the entire income of the Foundation fund for a year, supplemented by special gifts, together with the services of a Secretary. As soon as practicable after the armistice, National Research Council brought its war projects to a conclusion or made other disposal of them and created a tentative form of organization better suited to peace conditions, having thirteen divisions. There are six Divisions of General Relations, as follows: Government Division; Division of Foreign Relations; Division of States Relations; Division of Educational Relations; Division of Industrial Relations; and Research Information Service.

There are also seven Divisions of Science and Technology, as follows: Division of Physical Sciences; Division of Engineering; Division of Chemistry and Chemical Technology; Division of Geology and Geography; Division of Medical Sciences; Division of Biology and Agriculture; and Division of Anthropology and Psychology.

After three years' experience in an untrodden field, National Research Council is re-studying this form of organization, seeking greater economy and effectiveness.

From the beginning, cordial co-operation has been maintained between Engineering Foundation and National Research Council. The Secretary of each has been, by reciprocal arrangement, an Honorary Assistant Secretary of the other, the Secretary of the Foundation being thus a member of the Executive Board of the Council. Likewise, there has been exchange of office accommodations in New York City and Washington. Since the establishment of the Division of Engineering in March, 1919, it has been closely related to Engineering Foundation and since October of that year, the Foundation has provided offices in Engineering Societies Building for the Division, and a substantial portion of its funds. Measured on the financial basis, the work of the Division has been about 40% an activity of Engineering Foundation. In addition, the Foundation has contributed funds and services to a number of the research projects of the Division, besides undertaking other projects jointly with it. A certain number of members of the Foundation are, by definite arrangement, members of the Division. From October 1, 1920, to October 1, 1921, the Secretary of the Foundation was also a Vice-Chairman of the Division, and since October 1, 1921, he has been its Chairman.

The Division of Engineering formally states its objects to be: To encourage, initiate, organize, and co-ordinate fundamental and engineering research and to serve as a clearing house for research information in the field of engineering. Its total membership is 40, of which 21 are representatives from 14 National Engineering Societies, and one a liaison member of the Division of Federal Relations, and 18 members at large. In membership, it is more catholic than Engineering Foundation, since to the Founder Societies it has added ten other societies dealing with important engineering specialties. The work of the Division is conducted by its Chairman and Secretary, who are paid officers, a Vice-Chairman, and an Executive Committee having four additional members. There are four Advisory Boards, each dealing with a broad field of engineering, and four boards and fourteen committees, advising in limited fields or conducting research projects under the supervision of the staff of the Division.

The scope and importance of the work of the Division of Engineering, in which Engineering Foundation has so large a share, cannot be indicated briefly better than by the following list of the titles of its boards and committees:

Advisory Board on Civil Engineering Research, Robert E. Horton, M. Am. Soc. C. E., Chairman, sponsored by the American Society of Civil Engineers.

Advisory Board on Mining and Metallurgical Engineering Research, sponsored by the American Institute of Mining and Metallurgical Engineers.

Advisory Board on Mechanical Engineering Research, Walter Rautenstrauch, Chairman, sponsored by the American Society of Mechanical Engineers.

Advisory Board on Electrical Engineering Research, John B. Whitehead, Chairman, sponsored by the American Institute of Electrical Engineers.

Advisory Board on Testing Materials; the Executive Committee of the American Society for Testing Materials, George K. Burgess, Chairman.

Advisory Board on Illuminating Engineering Research; the Research Committee of the Illuminating Engineering Society, Edward P. Hyde, Chairman.

American Bureau of Welding, Comfort A. Adams, Director, sponsored by the American Welding Society. Committees on Electric Arc Welding, Welding Conference, Welding of Storage Tanks, Welding Wire Specifications, Standard Tests for Welds, Training of Operators, Specifications for Steel to be Welded, Resistance Welding, Thermit Welding, Welded Rail Joint Committee, Pressure Vessels.

Advisory Board on Highway Research, A. N. Johnson, M. Am. Soc. C. E., Chairman, W. K. Hatt, M. Am. Soc. C. E., Director; Committees on Character and Use of Road Materials, Economic Theory of Highway Improvement, Tractive Resistance of Roads, Structural Design of Roads, Maintenance, Highway Finances, Highway Traffic Analysis.

Committee on Deoxidizers, George K. Burgess, Chairman, J. R. Cain, Research Metallurgist.

Committee on Electrical Core Losses, A. E. Kennelly, Chairman.

Committee on Electrical Insulation, John B. Whitehead, Chairman.

Committee on Fatigue Phenomena of Metals, H. F. Moore, Chairman.

Committee on Hardness Testing of Materials, A. E. Bellis, Chairman.

Committee on Heat Treatment of Carbon Steel, F. B. Foley, Acting Chairman.

Committee on Marine Piling Investigations, R. T. Betts, Chairman, William G. Atwood, M. Am. Soc. C. E., Director; Sub-committees on Biology, Chemistry, Concrete, Finance, Engineering Patents, and Publicity.

Committee on Moulding Sand, R. A. Bull, Chairman; Sub-committees on Geological Surveys, Conservation and Reclamation, Standard Methods for Testing Moulding Sands.

Committee on Neumann Bands, Charles E. Munroe, Chairman.

Committee on Physical Changes in Iron and Steel Below the Thermal Critical Range, Zay Jeffries, Chairman.

Pulverizing Committee, Galen H. Clevenger, Chairman.

Committee on Uses of Tellurium and Selenium, Victor Lehner, Chairman.

Committee on Heat Transmission (in process of organization), H. C. Dickinson, Chairman of Organizing Committee.

Committee on Relation of Quality and Quantity of Illumination to Efficiency in the Industries. (Under consideration.)

These committees have from 6 to 49 members. The direct and indirect expenditures for projects in which the Division has some responsibility are estimated to exceed $2 000 000, but cannot be stated exactly, because many elements cannot be evaluated and accounts cannot be centralized for others, so as to be summarized, without expense and annoyance not justified by the result to be attained.

Except the committees in process of organization, all those named and several others which completed their projects some time ago and have been discharged, have achieved valuable results. For example, much has been added to the knowledge of the behavior of metals under repeated stresses; highway research has been co-ordinated, developed, and stimulated; more is

being learned about marine boring animals which destroy wooden piles and means for combatting them; the art of welding by gas and electric current has been notably advanced; and economies in the use of moulding sand are being effected. Some of these projects are being expanded in response to requests for further information. This is particularly true of the Fatigue of Metals Research to which Engineering Foundation contributed $30 000—its largest single appropriation—and to which the General Electric Company has contributed an equal amount and is adding to this investment. The Allis-Chalmers Manufacturing Company and other industrial corporations, also, are contributing.

Besides aiding in the support of the Division of Engineering and its projects, Engineering Foundation has had other activities. During the World War, it participated in experiments for spray camouflage of ships; a committee reported on the advisability of a laboratory for testing of large water-wheels and other hydraulic equipment of large size; and hollow-crested weirs were investigated under the immediate direction of Clemens Herschel, Past-President and Hon. M. Am. Soc. C. E. A study of the theory, design, and construction of arch dams has been recently undertaken as well as a study of wood-finishing processes in co-operation with the wood-working industries and users of paint and varnish on wood for buildings, vehicles, cars, furniture, and farm implements. An experimental study of reinforced concrete multiple arches is under consideration in co-operation with the Committee on Research of the American Society of Civil Engineers. Recently, Engineering Foundation was suggested as the best independent technical organization to take charge of the technical features of a concerted effort to overcome the cotton boll weevil in the Southern States. This project is under advisement on the understanding that ample funds will be provided by the American Cotton Association with which the proposal originated.

In addition to printing its annual reports for the past three years, Engineering Foundation has published a Progress Report on the Research in Fatigue of Metals, a Directory of Hydraulic Laboratories of the United States, and, since January, 1921, semi-monthly Research Narratives, 5-min. stories of interesting researches in many fields. Engineering Foundation has under consideration, as another project, a proposal for a continuing compilation of original engineering data taken from publications of the societies and technical journals, or contributed by individual engineers. For brevity, this project is known as the "Engineering Encyclopedia". As a result of the activities of Engineering Foundation and the Division of Engineering, numerous valuable papers and articles have been contributed to the societies and technical journals, and the research activities of the engineering societies have been stimulated.

The present income of Engineering Foundation seems small, as compared with the hundreds of thousands, and even millions, of dollars, spent annually for research by some of the most enterprising, and, consequently, prosperous, industrial corporations. The relatively large results mentioned in the foregoing paragraphs have been achieved through co-operation, supplemented by

the funds and efforts of the Foundation. As its resources increase, its services can be broadened to an approximation of the expectations of its many friends.

The plans of Engineering Foundation may be simply stated, as follows:

1.—To stimulate the spirit of research among engineers.

2.—To organize, further, or conduct researches or investigations which will benefit the Engineering Profession and the related industries.

3.—To aid in making results of research available to engineers, by contributions to publications of the societies and through other channels.

4.—To strengthen co-operation in research among engineers in various branches of the profession, between engineers and other technologists, and between engineers and scientists. (There are few, if any, researches connected with engineering or industry which do not involve such co-operation.)

5.—To serve as a clearing house of its member societies for research activities.

6.—To undertake especially such researches as cannot or will not be done by industries, Government, educational institutions, or separate societies, but to co-operate with all these agencies on proper occasions.

7.—To render service to individual engineers and to committees of the societies in all feasible and proper ways.

8.—To perform such broader services to mankind as fall distinctly to its lot, for which means are provided.

The greatest present needs of Engineering Foundation are:

1.—Substantial increase of endowment.

2.—More intimate co-operation of its Founder Societies.

3.—Such adjustment of relations with National Research Council as will leave the engineering part of the research field clear for Engineering Foundation.

4.—Provision for the participation of the other societies which are now members of the Division of Engineering.

Having created Engineering Foundation with Mr. Swasey's inspiration and financial aid, the Founder Societies seem straightway to have forgotten it, and have continued their research activities separately as if the Foundation did not exist. Here, however, is an instrumentality of great possibilities, belonging to the Founder Societies, which they should develop and utilize to their own advantage and credit, and "for the advancement of the Profession of Engineering and the good of mankind."

A liberally endowed Engineering Foundation, heartily supported by the great engineering societies, can seek the solution of many problems in engineering, or of mixed scientific and technical interest, that have practical application immediately in view. There is no other foundation in this field; each of the well-known foundations is devoting its resources to other subjects. There is a great opportunity for service.

THE STUDY OF STEELS FOR ENGINEERING STRUCTURES

By George K. Burgess,* Esq.

1.—Introduction

The subject of structural steel is one that cannot be treated adequately alone by either the civil engineer or the metallurgist. It requires the co-operation of both in best defining the material to be used for the several components of a given structural project, bridge, building, or ship, together with an understanding of relative manufacturing facilities and costs for the several possible materials, and the most economical solution, with sufficient provision for safety and permanence for the purchaser or the user.

The formulation of generally acceptable specifications naturally lags considerably behind the progress of the pioneers in the development of improved materials or methods of application, and the pioneer usually has first to take the risk himself in putting into practice those ideas the advantages of which he is convinced. The history of the development of steels for engineering structures is no exception to this rule.

Properly speaking, and looking forward as well as backward, this development of structural metals may be grouped into four periods, each designated by a class or grade of material: Wrought iron; low-carbon steel (at first, Bessemer and, then, basic open-hearth); alloy steels; and heat-treated steels, including high-carbon and alloy steels.

The wrought-iron stage gave way to that of Bessemer steel but gradually and, it might be said, with some misgivings; the gradation from Bessemer into basic open-hearth met no opposition, it was mainly a question of cost, convenience, and capacity of manufacturing, and Bessemer steel is still furnished for buildings. We are living in an epoch intermediate between the low-carbon steel and the alloy-steel periods. The great mass of structural steel tonnage is low-carbon, basic open-hearth steel, and many think it should remain so; nevertheless, not a few important bridges have been built in which alloy steels have been used, particularly of nickel and more recently of silicon and nickel-chromium, either in combination with carbon steels or alone; and, in a few instances, less important and smaller members of large bridges have been of weight-reducing heat-treated steels.

In addition to the type of steel to be chosen for the various parts of a structure, there are many auxiliary questions that must be considered and to some of which satisfactory answers are not as yet forthcoming, such as the relative behavior of thick and thin sections, of solid and fabricated members, and the best design for each; the limitations of riveting and the possibilities of welding; the allowable working stresses for each class of material, design, and load; the prevention of or protection against corrosion which is a different problem for structures exposed to atmosphere and for enclosed structures, such

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as city buildings, parts of which may also be subject to deterioration from stray electric currents; and, as a final illustration, the resistance to and protection from fire.

There is an important reason why the structural engineer should go slowly in accepting new ideas, and applying them. Many engineering structures are costly and they are usually built on a non-competitive basis as regards the engineering. The engineer, therefore, holds himself bound to follow accepted practice. For many types of structure, also, he is restrained by building codes and specifications of technical groups or societies that tend to crystallize existing practice.

In contrast to this enforced conservatism of the structural engineer, the freer position of the mechanical engineer, who also deals in metals, should be noted. Employed by the manufacturer, he is engaged in designing and distributing his product under keen competitive conditions and he is thereby stimulated constantly to improve it. The marvelous strides made in the knowledge and economical use of steels by the automotive industry which is now directing attention most effectively to aluminum and its alloys, need only to be mentioned to illustrate what progress is possible. The speaker wonders if any bridge engineer has ever considered whether he could advantageously use such an alloy as duralumin for any of the parts of a bridge? As compiled by Mr. H. C. Kneer,* Table 1 gives a comparison of the properties of duralumin and aluminum with those of mild and alloy steels.

TABLE 1.—Comparison of Properties of Duralumin and Aluminum with Steel.

	Duralumin tempered.	Aluminum half hard.	Mild steel.	Alloy steel.
Specific gravity	2.85	2.7	7.8	7.8
Weight, in pounds per cubic inch	0.010	0.094	0.27	0.27
Weight, in pounds of 1 sq. ft., 0.1 in. thick	1.44	1.35	3.90	3.90
Tensile strength, in pounds per square inch	55 000	18 000	55 000	125 000
Yield point, in pounds per square inch	25 000†	.....	36 000	100 000
Percentage of elongation in 2 in.	17	7	20	15
Modulus of elasticity, in millions of pounds per square inch	9.4-10	9.8	30	30
Ratio of strength to weight, $\frac{S}{W}$	19 250	6 650	7 050	16 000
Factor, $\frac{S}{W}$, compared, duralumin = 100%.	100	33.5	36.6	83
Sectional area for equal weight	1.00	1.05	0.36	0.36

† Values as high as 46 000 lb. per sq. in. have been obtained for yield point, with 15% elongation in 2 in.

The cost of duralumin construction will be found to compare favorably with heat-treated alloy steel, with an increase in strength efficiency of about 17% over alloy steel and 200% over mild steel. Is it possible that we may expect a fifth epoch in structural metals, that of aluminum alloys?

It may be of interest to note that the United States Bureau of Standards has tested about 143 duralumin girders for the new U. S. Navy dirigible, *Z-R1*.

* "Duralumin—A Digest of Information," *Transactions*, Am. Soc. for Steel Treating, Vol. 3, pp. 13–42 (1922).

These tests have not only given added confidence in the adequacy of these girders, but they have shown that there may be possibilities of lighter construction in future design.

For structural uses of duralumin, there are many factors other than lightness to be considered, such as its low modulus of elasticity—one-third that of steel—which will augment deflections under load; greater rigidity in tubes and channels; the elimination of the need of painting; lower transportation and erection costs; and the possibilty of greater permanence. There is a promising, immediate future for structural items and accessories for transportation units, but for important permanent structures the use of the light aluminum alloys to replace steels is still in the conjectural stage.*

2.—Structural Steel Specifications

As there will be occasion to refer to specifications frequently, it may not be out of place to record some of the specifications in current use for structural steels. In Table 2 are included typical specifications of the American Society for Testing Materials and some of the British Standards Committee, as well as a few steels which do not appear to have been endorsed by any specification-making body, but, nevertheless, have been used in important engineering structures. Among these steels are a so-called silicon steel and a nickel-chromium steel made from Mayari ores.

It will be noted that the British specifications permit somewhat higher tensile strengths, have no elastic limit requirements, and the ductility and chemical requirements are less severe than those of the American specifications.

As to the limitation of sulphur and phosphorus, attention might be called to the fact that it would appear to have been the general practice to frame specification limits for these two elements to meet what could readily be obtained by the current manufacturing practices for structural steel. Thus, the 1901 Specifications of the American Society for Testing Materials for buildings have no sulphur limit and permit phosphorus to 0.1% and in those for bridges and ships, the sulphur limit is 0.06% and the phosphorus limit is 0.08% for acid steel and 0.06% for basic steel; whereas the 1921 Specifications restrict the phosphorus in open-hearth steel to 0.06% for buildings, and for bridges the sulphur limit is 0.05% and the phosphorus limits have become 0.06 and 0.04 per cent. The present physical requirements, on the other hand, are somewhat less severe for steel for buildings and bridges than those of 1901. The requirements abroad as to sulphur and phosphorus for structural steel are less severe than the American requirements. Thus, the Unified French specifications of 1920 establish by definition† a maximum of 0.1% each for sulphur and phosphorus; the British specifications for bridges contain a maximum for each of these elements of 0.00%, and for general building

* See, also, "Some Points of Contact between Metallurgy and Engineering," by O. W. Ellis, *The Engineering Journal*, December, 1922, p. 576.

† Fascicule A1-2.

TABLE 2.—SOME STRUCTURAL STEEL SPECIFICATIONS.

Type.	No.	Process of manufacture.	Phosphorus, Maximum.		Sulphur, maximum.	Carbon, maximum.	Nickel, minimum.	Tensile strength, in pounds per square inch.	Yield point, minimum, in pounds per square inch.	Percentage elongation in 8 in., minimum.	Diameter on 180° cold bend, ¾ in.
			Acid.	Basic.							
A. S. T. M., bridges....	A 7-21	Open hearth	0.06	0.04	0.05	....	....	55 000- 65 000	0.5 Tensile strength	*27.3-18	1 T 2 T 3 T
A. S. T. M., nickel.....	A 8-21	Open hearth	0.05	0.04	0.05	0.45	3.25	85 000-100 000	50 000	*17.6-14 R.A.=25	1 T 2 T ...
A. S. T. M., buildings..	A 9-21	Open hearth Bessemer	0.06 0.10	0.06	....	....	....	55 000- 65 000	0.5 Tensile strength	†25.4-18	0 T 1 T 2 T
A. S. T. M., locomotives	A 10-21	Open hearth	0.05	0.05	0.06	....	....	55 000- 65 000	0.5 " "	*25.4-18	0 T 1 T 2 T
A. S. T. M., cars.....	A 11-21	Open hearth	0.06	0.04	0.06	....	....	50 000- 65 000	0.5 " "	*30 -18	0 T 1 T 2 T
A. S. T. M., ships.....	A 12-21	Open hearth	0.06	0.04	0.06	....	....	58 000- 68 000	0.5 " "	*26 -18	1 T 1½ T 2 T
Silicon	.....	Open hearth	0.06	0.04	0.05	0.40 Si. Mn.	0.20-0.45 < 1.00	80 000- 95 000	45 000	‡20 -16.8	1 T 1½ T ...
Mayari.....	.....	Open hearth	0.06	0.04	0.05	0.40 Ni. Si. Mn.	> 1.20 << 0.15 < 0.30	85 000-100 000	50 000	‡18.8-16 R.A.=30	.....
British:											
Bridges.....	15-1912	Open hearth	....	0.06	0.06	....	....	62 720- 78 920	....	20 -16§	1½ T
Buildings.....	15-1912	Open hearth	....	0.08	0.06	....	....	62 720- 78 920	.....	20- 16§	1½ T
Ships.....	13-1910	Open hearth	....	....	....	....	....	62 720- 71 680	.....	20- 16§	1½ T

*Based on $\frac{1\ 500\ 000}{\text{Tensile strength}}$.

†Based on $\frac{1\ 400\ 000}{\text{Tensile strength}}$.

‡Based on $\frac{1\ 600\ 000}{\text{Tensile strength}}$.

§For steel less than 0.375 in. in thickness.

construction, 0.08% for phosphorus and 0.06% for sulphur, whereas for structural steel for ships there appear to be no chemical limitations.*

Attention may be called to the work of the Joint Committee on the Investigation of Phosphorus and Sulphur in Steel composed of representatives of eleven organizations operating on the unanimous consent basis. This Committee has presented a Preliminary Report on the Effect of Sulphur on Rivet Steel† and this will be followed by other reports on the various grades of steel. It is expected that the results established by the investigations of this Committee will serve as a basis for the intelligent revision of the sulphur and phosphorus limits. The definite reports are to appear as *Technologic Papers* of the U. S. Bureau of Standards. An examination of the results obtained by the Committee, using material from fourteen 75-ton heats, on the properties of rivet steel (carbon = 0.11, manganese = 0.40 to 0.50, and phosphorus = 0.01), with sulphur in steps from 0.03 to 0.18%, shows that residual sulphur within this range appears to affect most of the physical properties only slightly, if at all. In general, the speaker thinks it will be found that high sulphur is of more concern to the manufacturer than to the user of steel.

3.—Modifications of Structural Steels

It has long been considered a desideratum to find an alloying element which can be added to structural steel at low cost and increase its strength and elastic limit without lowering the ductility and without resorting to special thermal or mechanical treatment. It has also been considered by some engineers that the present type of structural steel might be improved considerably by refinements in manufacture without essential change of composition or appreciable increase in cost, thus making for greater uniformity of product which might allow raising the permissible stresses. For certain structural members, also, the possibility of raising the carbon content somewhat at the expense of ductility has been considered, which would make for reduction in weight; and, finally, two or more of these suggestions have been combined, that is, raising the carbon and also adding an alloying element, such as nickel which tends to maintain the ductility, giving a satisfactory structural steel of considerably enhanced permissible stresses, say, 28 000 lb. per sq. in., as compared with 16 000 lb. per sq. in.

A well known factor that must be taken into account in structural steel when weight is decreased and stresses are increased, particularly for structures such as the larger bridges subject to heavy and variable live loads, is the greater deflection and consequent decreased stiffness undergone by the structure. These effects necessarily accompany the use of higher allowable stresses for a material like steel all the alloys of which show no change in the modulus of elasticity. In some cases, this factor may be counteracted by introducing strains of opposite sign during fabrication; and for each type and design of structure there is unodubtedly a best practice to be followed for a given material. The speaker will not discuss this phase of the question, but will confine himself to some of the metallurgical aspects.

* British Standard Specifications as of November, 1919.

† *Proceedings*, Am. Soc. for Testing Materials, Vol. 22, p. 94 (1922).

There are certain manufacturing characteristics generally common to all steels, but in diverse degree, such as piping, segregation, effect of impurities, including oxides and gas inclusions, internal stresses, and surface defects, but from which ordinary low-carbon structural steel may be and often is singularly free. With the increase of carbon, and especially with alloying elements in increasing quantities, it is more and more difficult and often practically impossible to make steel ingots free from pipe, and these more complex steels are also usually more subject to other defects, such as internal irregularities of composition and surface flaws, all of which results thereby in greater manufacturing losses, such as increased discard for pipe and rejection for variation in composition, properties, or condition of surface. In other words, the difficulties and cost of manufacture increase, as the complexity of the steel increases with, in general, a corresponding uncertainty in uniformity of the product, necessitating also a more rigid inspection.

Keeping these facts in mind, the modifications of structural steel by alloying and deoxidation, as well as the rôle of impurities, may be considered.

Among the alloying or addition elements besides carbon, that have been suggested, and used in some cases, are nickel, alone or together with chromium as for steel made from Mayari ores, vanadium, chrome-vanadium, silicon especially in association with higher manganese, aluminum which appears not to have been used except as a deoxidizer, and titanium. Other possibilities are copper and perhaps molybdenum.

It is interesting to note that, as early as 1904, the Pennsylvania Steel Company offered a special high-carbon steel for the structural work of the Manhattan Bridge, in New York City, to have an elastic limit of 45 000 lb. per sq. in., a tensile strength of 85 000 to 95 000 lb. per sq. in., and a percentage elongation in 8 in. of $\frac{1\ 500\ 000}{\text{tensile strength}}$; this steel had from 0.50 to 0.60% of manganese, and 0.45% of carbon, together with low silicon. The question naturally arises, may not a carbon steel of higher yield point than those enumerated in Table 2 be advantageously and safely used for structural purposes?

The properties of the steel mentioned may be compared with the latest specifications, those of the proposed Sydney Harbor River Bridge which has a central span of 1 600 ft. The specifications allow the contractor to use either carbon steel, nickel steel, or chrome-nickel steel. The specifications for plates and shapes up to and including 1 in. in thickness, are as given in Table 3.

Considering first the use of nickel in structural steels, the properties of 3.5% nickel-steel have been generally recognized for about fifteen years or more, the manufacturing conditions and costs are well established, and there are in existence widely accepted specifications such as the A8-21 of the American Society for Testing Materials (see Table 2). This material also has been advocated several times before the American Society of Civil Engineers, especially by J. A. L. Waddell, M. Am. Soc. C. E. Carbon is allowed up to 0.45% in this steel which has a yield point (50 000 lb. per sq. in.) nearly double that of low-carbon structural steel, but with slightly less ductility.

TABLE 3.

	Carbon steel.	Nickel steel.	Chrome-nickel steel.
Minimum tensile strength, in pounds per square inch	62 000–70 000	85 000–100 000	85 000–100 000
Minimum yield point, in pounds per square inch	35 000	50 000	50 000
Percentage of elongation in 8 in	1 500 000 / Tensile strength	1 600 000 / Tensile strength	1 600 000 / Tensile strength
Minimum reduction of area, percentage	44	40	30

This material was first used in bridge construction about 1904 in the Blackwell's Island Bridge designed by Gustav Lindenthal, M. Am. Soc. C. E. Mr. Waddell has advocated insisting on a yield point of 55 000 or even 60 000 lb. per sq. in. for this grade of steel, but it should be borne in mind that if the chemical composition is kept constant, the yield point can be raised only by thermal or mechanical treatment (as by cold working it), a procedure which tends to make for lack of uniformity of properties. This implies a best value for the finishing temperature of rolling any given section of a material. It may be that a yield point of 50 000 lb. per sq. in. is too low for this material, but the specified tensile strength and ductility are probably also low. This material as used in the construction of the St. Louis Bridge,* for unannealed specimens averaged an ultimate strength of 99 850 lb. per sq. in., a yield point of 60 250 lb. per sq. in., an elongation of 17.9% in 8 in., and a reduction in area of 33.3%, with a chemical composition, as follows:

Carbon	0.38 per cent.
Manganese	0.58 " "
Nickel	3.45 " "
Phosphorus	0.012 " "
Sulphur	0.03 " "

In the Manhattan Bridge, about 8 000 tons of plates and shapes of nickel steel were used, as well as nickel-steel rivets. The requirements were tensile strength, 85 000 to 95 000 lb. per sq. in., yield point, 55 000 lb. per sq. in., and percentage of elongation in 8 in. of $\frac{1\ 600\ 000}{\text{tensile strength}}$. Nickel steel has also been used in the following bridges: The new Quebec Bridge, the Frott Bridge in Kansas City, Mo., the Jack-knife Bridge in Chicago, Ill., and the Detroit-Superior High Level Bridge, in Cleveland, Ohio.

The element that may best be compared with nickel for addition to structural steel is copper, which has been shown to increase the strength, yield point, and resistance to shock and to atmospheric corrosion, without loss of ductility. Experimenters have disproved the erroneous idea of embrittlement of low and medium carbon steel by copper.† A steel containing 0.38% carbon, 0.57% manganese, and 0.86% copper, will have a yield point near that of

* "Bridge Engineering," by J. A. L. Waddell, p. 91 (1916).

† See paper by Messrs. Hayward and Johnston, *Transactions*, Am. Inst. of Min. and Metallurgical Engrs., Vol. LVIII, p. 722 (1918), and references therein.

3.5% nickel steel, with considerably greater ductility. In the tests of Messrs. Hayward and Johnston, the elongation in 2 in. was 27.3% and the reduction in area 52.7%, as compared with the requirements of nickel steel of 16 and 25%, respectively, in Specification No. A8-21 of Table 2.

Messrs. Clevenger and Ray* have noted the tendency of copper to segregate toward the bottom of small ingots when more than 0.8% is added as metallic copper. The relatively low cost of copper and the ease with which it may be added to structural steel indicate that it should be recommended, at least after trial in some large heats on full-sized structural members. In fact, this appears to be a most promising field of research from the points of view of engineering, metallurgy, and economy, and it would probably be more than worth while to conduct an investigation to determine the best metallurgical practice, manufacturing costs, and most suitable limits for copper, either alone or with other elements, such as nickel and manganese, in structural steel, similar to that reported by Mr. Waddell† for nickel steels. It may be recalled that some American iron ores contain copper that enters and remains in the steel which often contains 0.25% copper to its improvement. It is possible that there might be advantage in using either the copper-nickel ore from which monel metal is made, for example, or some intermediate product, or even monel metal itself (68% nickel, 28% copper, the residual being mainly iron and manganese), in producing an economical copper-nickel structural steel.

The case for vanadium as an alloying element has been attractively stated by Mr. G. L. Norris,‡ although, as Mr. Waddell pointed out, this exhibit was not without ambiguity, either from the metallurgical or cost point of view. The advantages of vanadium appear to be greatest in association with chromium for steels to be heat treated.

An interesting possibility is the greater development of nickel-chromium structural steels, using such ores as the Mayari as a base. Large steel columns made from this steel containing from 1.12 to 1.58% nickel and 0.37 to 0.66% chromium were tested at the U. S. Bureau of Standards for the Metropolis and Memphis Bridges.§ Some of these steels also had a copper content as high as 0.1 per cent. Credit is due to Ralph Modjeski, M. Am. Soc. C. E., for the first use of Mayari steel in bridge construction, namely, for the Memphis Bridge. A large number of smaller railway bridges have since been constructed in which Mayari steel has been used.

Another alloy steel that is coming into use for structures, such as parts of the Delaware Bridge, is the so-called silicon steel (Table 2) which might perhaps as well be named a silicon-manganese steel as the specifications call for a maximum of 0.40% carbon, 0.45% silicon, and 1.00% manganese, with a minimum of 0.20% silicon. The raising of the elastic limit by increase of silicon seems to be of English origin. The Sandbergs urged the use of 0.3 to 0.4% silicon in rail steel about 1903. Silicon steel appears to have been first

* "The Influence of Copper upon the Physical Properties of Steel," *Bulletin*, Am. Inst. of Min. Engrs., No. 82, pp. 2437–2475 (1914).

† "Nickel Steel for Bridges," *Transactions*, Am. Soc. C. E., Vol. LXIII (1909), p. 101.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 72.

§ *Technologic Paper 184*, U. S. Bureau of Standards.

produced in America about 1908 or 1910 by the Cambria and Carnegie Steel Companies in the form of plates and shapes for light torpedo-boat destroyers, for which the U. S. Navy Department desired a steel having a tensile strength of 80 000 to 90 000 lb. per sq. in., and a steel having approximately 0.75% silicon, 0.60 to 0.80% manganese, and 0.30 to 0.40% copper, was produced for that purpose. In 1912, the Carnegie Steel Company, the speaker understands, made the first proposal of a silicon steel for bridge construction, for the eye-bars and structural steel of the Memphis Bridge.

Here, again, is a case in which a so-called impurity, silicon, has become a desirable and relatively cheap alloying element, imparting strength and high elastic limit to the steel. The present specification, however, is really a double one, permitting the physical properties desired (Table 2) to be attained by either high silicon or high manganese. The manufacturers naturally will furnish a steel of an adjusted composition, depending on the relative price of ferro-silicon and ferro-manganese. Steel of this kind was also tested at the U. S. Bureau of Standards in the form of large columns for the Metropolis Bridge.* The Bureau is now making additional tests of this steel made into fabricated parts for the Delaware Bridge.

The rôle of titanium has been well described by Mr. N. Petinot† to be primarily that of a deoxidizer and scavenger and an effective agent for reducing segregation. It is usually added with the ladle additions in the form of ferro-carbon-titanium containing 16% titanium, about 8 to 12 lb. of ferro-carbon-titanium per ton of steel (0.06 to 0.08% titanium), and there remains in the steel less than 0.02% titanium.

The U. S. Bureau of Standards is completing an investigation of the manufacture and properties of two 1 000-ton lots of rail steel (0.75% carbon and 0.76% manganese) treated with titanium, as compared with similar steel deoxidized with ferro-silicon. The mechanical properties of these two steels are not markedly different, but the titanium-treated steel has less segregation in the upper part of the ingot and shows a slightly greater tendency toward piping. One would expect still less differences for low and medium carbon steels.

With regard to the possibilities of improving the physical properties of ordinary structural steel—yield point and ductility—by endeavoring to produce steel more highly "purified", it is questionable whether much is to be gained by this means. A series of experiments‡ just completed at the U. S. Bureau of Standards on the mechanical properties of pure alloys of iron, carbon, and manganese, containing no phosphorus, with less than 0.01% sulphur and silicon for most of the samples, show that, although there is a slight improvement in ductility, there is a corresponding decrease in strength and elastic limit as compared with commercial steels of the same carbon and manganese contents.

In view of the almost negligible importance of the phenomena of fatigue for most structural steels of interest to the civil engineer, the question of freeing the steel still further from entrained inclusions, although it may be

* *Technologic Paper 184*, U. S. Bureau of Standards.

† *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 50.

‡ *Technologic Paper 453*, U. S. Bureau of Standards.

said to be desirable, appears not to be vitally essential, and probably would not allow raising the permissible stresses, whereas the cost would be increased. Structural steel is much less subject to abuse in service than rail steel, for example, and the manufacturing practices permitted by the specifications for rail steel do not necessarily result in clean steel. This should not be taken as an argument for dirty steel, but it must always be borne in mind that for any engineering material there is a most economical procedure in manufacture to meet the engineering requirements. The manufacturer, in general, can meet more rigid requirements, but not without increased costs, except as his art is improved.

Evidently, there are possible uses of heat-treated steels in engineering structures, and such steels have been introduced to a limited extent in lattices and auxiliary parts. The difficulties of furnishing a reliable and uniform product increase rapidly with the size of heat-treated pieces, as well as the difficulties and costs of joining by rivets or otherwise. For certain structural purposes, as parts of bridges, the line of least resistance would seem to be the development, first, of higher carbon steels which may be air-quenched by some method, such as the Sandberg process. After a study has been made of the most economical alloying element or combination to give maximum yield point without serious loss of ductility in steel to be air-treated, it may be possible to consider liquid-quenched and tempered steels, although, in the beginning, these steels should be limited to small parts.

Mr. Modjeski appears to have been the first to have used heat-treated eye-bars for bridges. For the Memphis Bridge, he used Mayari steel for which the rather mild treatment consisted in an anneal at about 1 500° Fahr., followed by an air-quench. These heat-treated eye-bars, as specified by Mr. Modjeski, and as furnished by the Pennsylvania Steel Company, had the properties given in Table 4.

TABLE 4.

	Specified.	Furnished.
Minimum tensile strength, in pounds per square inch	80 000	90 000
Minimum elastic limit, in pounds per square inch	47 000	56 000
Minimum elongation, percentage	10	13

This composition was approximately, as follows:

Carbon	0.35	per cent.
Manganese	0.70	" "
Nickel	1.40	" "
Chromium	0.45	" "

It is questionable whether it will be found economical or safe to urge the use of severely heat-treated alloy steels for the main members of engineering structures, and, at present, there are few manufacturing concerns equipped, or with adequate experience, for such production. In parts of structures

subject to fire hazard, the use of heat-treated struts would be an additional hazard as, in general, they lose their enhanced properties with rise of temperature.

4.—Problems in the Use of Steels for Structures

No pretense is made that this list of problems is complete. It is limited to a brief consideration of some of those of which the Bureau of Standards has contributed to the solution. Although of apparently great diversity, they all have to do with the limitations of the use of structural steels.

Fire-Resisting Qualities.—In certain types of engineering structures subject to fire hazard, such as buildings, it is of prime importance to know accurately the behavior of steel at high temperatures. It is also of interest to determine whether it is possible to develop a structural steel which maintains its strength at high temperatures, as well as the safe temperature limits, and also the most effective method of protecting steel columns against fire.

The determination of the mechanical properties of various steels with changes in temperatures up to 650° cent. (1 200° Fahr.) is being conducted at the U. S. Bureau of Standards by Mr. H. J. French. The results have been published* on the Effect of Temperature, Deformation, and Rate of Loading on the Tensile Properties of Low-Carbon Steel (0.17 to 0.25% carbon, 0.40% manganese) and on the Tensile Properties of Some Structural Alloy Steels at High Temperatures.†

It may be stated in brief that a low-carbon steel has its maximum strength between 250 and 300° cent. (480 to 570° Fahr.) and minimum ductility throughout a somewhat wider range. Above 300° cent. (570° Fahr.), the strength decreases rapidly, accompanied by rise in ductility, whereas the elastic limit begins to decrease at a somewhat lower temperature than the strength. At 450° cent. (760° Fahr.), the tensile strength becomes about two-thirds of its value at normal temperature, and the elastic limit from one-half to two-thirds this value. This material moderately cold-worked by rolling maintains the imparted properties of increased elastic limit and slightly diminished ductility to a temperature of 465° cent. (870° Fahr.).

Tests made at various high temperatures on a number of carbon and alloy steels indicate that above about 550° cent. (1 020° Fahr.), the strength of all steels is low and decreases rapidly with increase in temperature. In the higher temperature ranges, the metal is plastic and will continue to deform as long as the load is maintained, and, finally, will fail.

It is improbable that any steels can be produced to stand continuously or for reasonably short intervals appreciable loads at temperatures above about 650° cent. (1 200° Fahr.), except in cases where large proportions of alloying elements are added to reduce the iron content to such low values that the resulting product cannot correctly be called steel.

However, at temperatures near 450° cent. (760° Fahr.) and less, it is possible to improve the strength of ordinary structural steels by additions of such elements as chromium, cobalt, and uranium, and less effectively with

* *Technologic Paper 219*, U. S. Bureau of Standards.
† *Technologic Paper 205*, U. S. Bureau of Standards.

nickel, or combinations of these and other elements, or by heat treatment, the relative strength at the high temperature of the different combinations being roughly proportional to the properties at room temperature.

There would appear to be no economic advantage, therefore, except possibly in some very special cases, of modifying structural steel to resist high temperatures. This opinion is greatly strengthened by the investigation of Fire Tests of Building Columns,* recently conducted by Mr. S. H. Ingberg and his associates as a joint problem of the U. S. Bureau of Standards, the Underwriters Laboratories, and the Associated Factory Mutual Fire Insurance Companies.

The purpose of this investigation was to ascertain (1) the ultimate resistance against fire, of protected and unprotected columns as used in the interior of buildings; and (2) their resistance against impact and sudden cooling from hose streams when in a highly heated condition. The series consists of tests of 106 columns, of which 91 were fire tests and 15 fire and water tests.

The fire-test series include (1) tests of representative types of unprotected structural steel, cast iron, concrete-filled pipe, and timber columns; (2) tests wherein the metal was partly protected by filling the re-entrant parts or interiors of columns with concrete; (3) tests wherein the load-carrying elements of the columns were protected by 2 or 4-in. thicknesses of concrete, hollow clay tile, clay brick, gypsum block, and also a single or double layer of metal-lath and plaster; and (4) reinforced concrete columns with 2-in. integral concrete protection.

The test columns were designed for a working load of approximately 100 000 lb., as calculated according to accepted formulas, the amount varying somewhat for the different sections. The load was maintained constant on the column during the test, the efficiency of the column or its covering being determined by the length of time it withstood the combined load and fire exposure.

The column was exposed to fire by placing it in the chamber of a gas-fired furnace the temperature rise of which was regulated to conform with a pre-determined time-temperature relation. Measurements were taken of the temperature of the furnace and test column and of the deformation of the column due to the load and heat.

In the fire and water tests, the column was loaded and exposed to fire for a pre-determined period, at the end of which the furnace doors were opened and a hose stream applied to the heated column, the duration of the application and the pressure at the nozzle varying with the length of time the corresponding type of column withstood the regular fire tests.

The steel for the seventy-seven steel columns was basic open-hearth, conforming within 10%, above or below, the specifications of the American Society for Testing Materials for structural steel for buildings (Table 2) and the applied working loads averaged 11 600 lb. per sq. in.

The time to failure of the unprotected structural steel columns varied from 11 to 21 min. The difference in results for the various column types, although in part due to variation in furnace exposure, is attributable also to difference in thickness of metal and in the unit loads sustained, sections with thin

* *Technologic Paper 184*, U. S. Bureau of Standards.

members under the higher unit loads failing more quickly than sections the members of which were arranged to form heavy metal thickness.

The average time to failure of the eight structural steel sections was 15 min., and the average period of expansion was 13 min. Maximum temperatures from 578 to 668° cent. (1 072 to 1 234° Fahr.) were attained on the outside of the metal near the edges. It is difficult to estimate the average effective temperature, as the rise was too rapid to permit assuming temperature uniformity over the thickness of the metal. Tests recently made at the U. S. Bureau of Standards on small specimens* indicate that, for the loads sustained by the unprotected columns, the failure temperatures of the structural steel fell within the limits of 550 to 650° cent. (1 022 to 1 202° Fahr.). The failure in the fire test in all cases was due to decrease in mechanical strength of steel with increase in temperature.

Apparently, it appears hardly profitable to secure a little greater fire resistance of structural steel by alloying or treatment, as heat-insulating protection would still be necessary, unless greater fire resistance could be attained incidentally with greater general strength and uniformity. If, however, the working unit stresses are increased in proportion to the resulting increase in elastic limit or ultimate strength, the gain for fire resistance would probably be small, as it decreases with increase of load imposed.

In contrast to the quick response of only a few minutes of unprotected steel to fire conditions, it may be noted that, for the reinforced concrete columns tested, the time to failure averaged more than 8 hours.

Expansion.—A knowledge of change in length with rise in temperature is of considerable importance in the design of certain structures. The U. S. Bureau of Standards is equipped for determining the thermal expansion at high and low temperatures, and the expansion coefficients have been measured† by Messrs. Souder and Hidnert for a number of steels, a few of which are indicated in Table 5, from which it is seen that, except for very unusual compositions, the expansion coefficients of most steels do not differ greatly.

Painting Structural Steel.—The question of the protection of Structural steel by paint has received considerable attention at the U. S. Bureau of Standards and specifications have been published, some of which are among the first to be promulgated as United States Government specifications by the recently organized Federal Specifications Board.

After thorough mechanical cleaning to remove rust and scale, the Bureau recommends protecting structural steel with at least two and preferably three coats of red lead, linseed oil paint. This red lead paint should be mixed on the job and used within 24 hours after mixing. Directions for mixing red lead for three-coat work on steel, tinted so as to give a final dark brown color, are given on page 79 of *Circular 69,* Paint and Varnish. Specifications for red lead are given in *Circular 90,* for linseed oil, in *Circular 82,* for turpentine, in *Circular 86,* and for drier, in *Circular 105.*

For additional color coat, paints meeting the specification of *Circular 94,* United States Government specification for Black Paint, Semi-paste and

* *Technologic Paper 205,* U. S. Bureau of Standards.
† *Scientific Paper 433,* U. S. Bureau of Standards.

Ready Mixed, *Circular 93*, United States Government specification for Iron-Oxide and Iron-Hydroxide Paints, *Circular 89*, United States Government specification for White Paint and Tinted Paints Made on a White Base, Semi-paste and Ready Mixed, and *Circular 97*, United States Government specification for Green Paint, Semi-paste and Ready Mixed, may be used. Expense may be saved with no very great decrease in the quality of the job by using one of these paints in place of the third red lead coat, but other paint should not be substituted for the first and second red lead coats.

TABLE 5.—STEELS: AVERAGE COEFFICIENTS OF EXPANSION ON HEATING.*

Composition.						25 to 100° cent.	25 to 300° cent.	300 to 600° cent.	25 to 600° cent.
Carbon.	Manganese.	Silicon.	Nickel.	Chromium.	Copper.				
0.02	0.00	0.006	Co. Cu. Ni. total 0.014			12.0	13.3	15.9	14.7
0.252	0.06	0.007	.....	.....	.....	11.1	12.5	16.0	14.3
0.41	0.64	0.086	.....	.....	.....	11.1	12.7	15.8	14.3
0.388	1.21	1.04	3.67	.....	.....	11.6	12.3	15.0	13.7
0.326	0.78	0.094	3.59	.....	.....	10.9	12.1	15.3	13.8
0.168	0.01	0.135	3.94	2.50	$V = 0.39$	10.8	12.1	14.3	13.3
0.35	1.42	0.20	.....	1.00	$V = 0.11$	12.4	13.3	15.6	14.5
0.67	0.77	0.19	.....	.....	.....	11.7	13.0	....	....
0.144	0.10	0.034	$V \pm 0.21$	1.15	1.85	11.2	12.7	15.7	14.3
3.08	(?)	1.68	.....	.....	.....	8.4†	11.6†	Grains	
0.14	(?)	(?)	34.52	.....	.....	3.7	9.2	18.2	13.6
0.35	(?)	(?)	.....	13.0	.....	10.0	11.0	13.3	12.2

* The values of the coefficients given in Table 5 should be divided by 10^6.
† Approximate only, as this material increases permanently in length.

Electrolytic Deterioration.—From time to time there have been reports of serious damage to structural steel, particularly reinforcing material in concrete, in the footings of buildings, bridges, elevated railways, and similar structures. A few years ago, this hazard was thought to be serious and, in consequence of this fear, Mr. Burton McCollum and his associates at the U. S. Bureau of Standards made an exhaustive research of electrolysis in concrete and electrolytic damage to bridges, steel frame buildings, and similar structures. The net result of this investigation was that in very extreme cases some damage might occur from this source, but that the conditions necessary for any serious damage to result were far more extreme than those likely to be found in practice. Many reports of alleged damage to such structures were carefully investigated, and it was found that in all cases such damage was due to causes other than electrolysis, with the single exception of the corrosion of the bases of elevated railway structures where, for obvious reasons, electrolytic conditions were particularly severe. If the most obvious and elementary precautions are taken, such as avoiding metallic connection with power circuits, in the case of the building foundations, and of metallic connection with rails, in the case of elevated railway structures, no trouble from this source is to be anticipated.

Failures.—One of the important functions of the U. S. Bureau of Standards is the study of metal failures. As illustrations in the field of structural metals may be mentioned the examination of the parts of a railroad bridge and

of a roof truss. Such examinations include chemical analysis, tests of mechanical properties, and a detailed metallographic study of the material.

The bridge material, submitted by the Board of Railroad Commissioners of the State of Iowa, was of wrought iron from a diagonal member of a single-system, Warren truss bridge of 85 ft. span, that failed under the load of a locomotive of the Pacific type and a tank car. A study of the failed parts showed inferior material laminated to an excessive degree, with evidences of long continued over-stressings of the material.

The roof truss was taken from the Knickerbocker Theatre after the disaster at Washington, D. C., and submitted by the Building Inspector of the District of Columbia. The material was taken from Trusses 11 and 12, one of which certain of the interested parties had claimed to be of defective material. The tests at the U. S. Bureau of Standards showed this material to be well within the specifications for this class of structural steel and to be free from structural features and inclusions which would lead one to regard the steel as defective. Chemical analysis showed carbon, 0.12%, manganese, 0.55%, phosphorus, 0.014%, sulphur, 0.046%, and silicon, 0.03%; the mechanical properties of the several parts ranged as follows: Tensile stress, 51 200 to 56 100 lb. per sq. in.; yield point, 33 200 to 36 300 lb. per sq. in.; elongation in 8 in., 30 to 36%; reduction in area, 58 to 64%; Charpy impact value, 105 ft-lb.; and Izod, 79 ft-lb. Thus, this material apparently failed from an external cause, such as a blow during the collapse of the structure.

Reinforcement Bars.—Considerable attention has recently been given by specification-making bodies, such as the American Society for Testing Materials and the testing laboratory at the U. S. Bureau of Standards under the direction of W. A. Slater, M. Am. Soc. C. E., to the question of the determination of the characteristics of concrete reinforcement made from salvaged steel, such as re-rolled rails. Such an investigation is of great practical importance in view of the large available quantity of such material and the fact that its use is quite generally prohibited at present, on account of alleged brittleness, causing breakage in handling, and the uncertainty of its behavior in structures.

The U. S. Bureau of Standards has made tests on bars purchased from a number of warehouse stocks. From these tests, it appears that either re-rolled or new billet steel with yield points up to 60 000 lb. per sq. in., which meet the tension and bend tests, would be satisfactory for concrete reinforcement. From these tests, it appears doubtful whether steel having a yield point greater than 60 000 lb. per sq. in. can be bent to meet conditions in practice, but reinforcement that does not require bending, should be satisfactory if it meets the tension test and suitable impact or bending tests to detect brittleness. The brittleness appears to be associated more with the yield point of the material than the treatment of the bars in re-rolling them. It is important that this subject be investigated adequately in view of the possibility of salvaging large quantities of old rails by re-rolling for use in reinforcement bars.

This question is closely linked with the subject of deformed bars and allowable bond stresses between concrete and steel. Some types of deformed bars develop weakness on bending, due to more or less sudden changes in area of section. Present practice tends toward elimination of sudden changes in

section for that reason, but the efficiency of a bar in resisting the tendency to slip is involved in this to an important degree. Tentative standards of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete permit a bond stress 20% greater for an "approved" deformed bar than for a plain bar, but the Committee has arrived at no agreement as to the shapes of deformed bars which it approves. It has not even agreed on the form of test specimen which will give the proper comparison between plain and deformed bars.

The divergencies of opinion are sharp on the questions involved in all the reinforcement bar problems and further extensive investigation of the entire problem is imperative.

Permissible Stresses for Steel in Buildings.—If the cost of steel structures is to be lowered, it can be done most effectively by increasing the stress and using the material more efficiently.

The increase from 16 000 to 18 000 lb. per sq. in. for the new Hotel Statler in Buffalo, N. Y., is a move in that direction, which is receiving considerable consideration. If steel is to hold its own in cost with other structural materials, particularly reinforced concrete, the stresses must be increased.

Mr. H. L. Whittemore, of the U. S. Bureau of Standards, has made a study of the present status of this subject, based on ordinary structural steel, taking account of conclusions drawn from various tests on solid and fabricated columns, made at the Bureau and elsewhere, as well as present practice as illustrated in manufacturers' and engineering handbooks and as formulated in about sixty-three municipal and other building codes, including also some foreign specifications and those proposed by various American committees and prominent structural engineers. It is expected that this survey will be published soon in full.

In that study, emphasis is laid on the necessity for inspecting and testing structural steel if more than one quality is commonly used. This and the obtaining of higher strength would increase the cost of the steel somewhat. The same results might be obtained by designing structures for the actual maximum loads they would carry. Thus, a lightly loaded residence would require periodic inspections to prevent its being overloaded if a change in the surroundings made it desirable to use it for manufacturing purposes.

If the load is constant and its value known, it is probable that stresses near the yield point could occur before failure. The greater factor of safety required by most city building codes is made necessary by the uncertainties of the properties in the material and the load which will actually occur. Nearly all codes for American cities limit the stresses to 16 000 lb. per sq. in. The maximum stresses in long columns $\left(\frac{l}{r} = 90\right)$, as given by any formula or any building code (Buffalo, N. Y.), is 11 970 lb. per sq. in. There has been no tendency to increase this limit in recent years, but rather to decrease it for large ratios of $\frac{l}{r}$.

Even German specifications, otherwise more liberal than American specifications, are very conservative on this point. Considering the damage which

may result from the failure of a column and the uncertainties in regard to actual loading, workmanship, and uniformity of the material, it is believed by Mr. Whittemore that the values given by the formula proposed by the Special Committee on Steel Columns and Struts,* namely $S = 20\,000 - 100\,\frac{l}{r}$, should not be exceeded. Possibly higher stresses than the limiting value of 12 000 lb. per sq. in., given by the Committee, might be used for short columns $\left(\frac{l}{r} = 40, \text{ etc.}\right)$.

In considering this matter, it should also be borne in mind that it appears to be the opinion of some of the most eminent American engineers that it is safe to raise the permissible stresses for steel in buildings from 16 000 to 18 000 lb. per sq. in., if precautions are taken to check the grade of steel and inspect thoroughly its erection, with limitations on the increase of the live load. This subject, of such great complexity and economic importance, is now in the hands of the Special Committee on Stresses in Structural Steel of the Society. The speaker ventures the suggestion, which, perhaps, has already been considered, that procedure under the rules of the American Engineering Standards Committee, with the Society as sponsor, would be a most efficacious method of securing co-operation of all interested bodies and of establishing a standard American practice for each class of structure and material.

Riveting and Welding of Structures.—The scope of this paper will permit only a passing reference to these important subjects. Attention might again be called to the report of the Joint Committee on the Effect of Sulphur on Rivet Steel (Section 2). Mention has also been made (Section 3) of the early use of nickel-steel rivets in bridge construction, which permits rivets of smaller section. In spite of the apparent economy and advantages to be secured, their use does not seem to have increased to the extent that might have been expected. There are undoubtedly outstanding factors yet to be considered; thus, the nickel-steel rivet is somewhat harder at the temperature at which it is driven and may fail to fill the hole as thoroughly as a carbon-steel rivet. It is also more difficult to cut out cold nickel-steel rivets, owing to their greater hardness and toughness. It would seem that these difficulties were not hard to overcome; thus, use of the newer electric rivet-heating furnace would enable the operator to produce a higher temperature and, consequently, the softness necessary to drive the rivet satisfactorily, and the burning-off method could readily be used to remove cold rivets.

An important problem in engineering design planned to be conducted at the U. S. Bureau of Standards for the U. S. Navy Department concerns the strength of riveted connections, and is of especial interest not only for the completeness of the contemplated test schedule and methods of test, but also because the riveted joints to be tested are the largst specimens thus far suggested. The test specimens will be 22 in. wide, and 13 ft. long, will contain as many as twenty 1-in. rivets on each side of the plate joint, and with supple-

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1583.

mentary pulling grips will take up the full length of the large Emery testing machine, as well as require its full capacity to produce joint failure.

Thorough and precise exploration of stress distribution, of frictional failure, and joint strength are contemplated, and it is further hoped to secure valuable data on the following points:

(*a*) The effect of plate material on rivets. (Three grades of steel plates, mild steel, high strength steel, and special treatment steel, the latter possibly a nickel alloy steel and two grades of rivet steel are now proposed).

(*b*) Effect of rivet size ($\frac{7}{8}$-in. and 1-in. rivets).

(*c*) Effect of the increase in the number of rows of rivets on joint strength and the distribution of the load between the rows of rivets.

(*d*) Effect of the omission of alternate rivets in the outer row.

(*e*) Study of the strength of a special form of joint.

(*f*) Study of the strength of joints in two-course plating.

The joints will be tested in triplicate and the present contemplated program calls for sixty-six specimens.

Rapid progress is being made in the welding of steel structures, about twenty or thirty structures of note having been constructed, some of which are very large. According to F. P. McKibben, M. Am. Soc. C. E., most of them have not been very carefully designed with welding in view. Many, in fact, were designed as riveted structures, but were changed by shopmen to be electrically welded. The development of stresses due to welding is not comparable with the stress conditions of riveting, and suitable allowance should be made in design and erection to provide for these differences. There is also need for much more experimental work in the testing of welded joints. Professor McKibben states that his examination of welded steel structures shows that the tension tests, as ordinarily made on butt welded specimens, are not of much value in giving adequate data to designers. Tests appear to be needed on the shearing strength of welded joints.* The art of welding is still in its infancy and a great deal is yet to be done in the improvement of the technique of the various methods and in the choice of welding rods to produce welds in which the added metal is of satisfactory quality. The U. S. Bureau of Standards has done some work along these lines, particularly with reference to studies of gas-welding equipment† and the study of gas content of welds as affecting physical properties,‡ and the American Bureau of Welding is actively conducting an extensive research program covering many problems in the development and use of welding.

A problem relating to structures in the comparison of welding and riveting is being studied at the Bureau with the American Bridge Company which has been experimenting with a new process of resistance electric welding, making use of pressed or inserted buttons to localize the welding action. About 150 comparative tensile tests of plain plate-riveted and welded joints were made. The welded joints were from 2 to 23% stronger than the riveted joints, the

* See article by A. S. Humphreys, Jr., in *Iron Age*, May 25, 1922.

† *Technologic Paper 200*, U. S. Bureau of Standards.

‡ *Technologic Paper 179*, U. S. Bureau of Standards.

average being about 15 per cent. In no case did the welded joint show a lower strength than the corresponding riveted joint.

In addition, comparative tests were made on three riveted and three welded girders in cross-bending, and three riveted and three welded columns, with slenderness ratio of 120, with flat ends, under axial load. None of these girders failed due to either rivet or weld failure. The welded girders averaged 1% stronger than the riveted girders and the welded columns average 13% weaker than the corresponding riveted columns. Supplementary coupon tests showed that this difference could not be ascribed to differences in the material. In all cases, the welded girders deflected more under load than the corresponding riveted specimens, the average being 6 per cent. It is highly probable that the differences in column strength are due to this greater flexibility which could be overcome by more closely spaced or larger welds.

The general conclusion is that, in structural members, welding shows sufficient promise of becoming an economical substitute for riveting to warrant the extensive series of tests which would be necessary to establish the limitations of its usefulness.

5.—Experimental Studies of Structural Parts

In the structural engineering field there are many unsolved problems concerning the distribution of stress in, and the strength of, various structural members and their connections. Many of the rules governing designs are based on the results of theoretical analysis, or exchange of thought by discussion, or are the summation of constructional experience. Some of these rules are obviously empirical, others have been founded on relatively meager experimental data, and many of them lack experimental proof. Engineers do not always know, therefore, whether they are just within the safe limit in their practice or whether they are far too conservative. As the magnitude and importance of structures have increased, the rules, at times, have proved inadequate. At other times, some of the limitations accepted being the relative economy of various designs to the fire, decisions of the latter type need to be verified by data in order to allow correct appraisal.

The equipment available for investigations in this field at the U. S. Bureau of Standards consists of an Olsen vertical machine of 10 000 000 lb. capacity, in which columns up to 24 ft. in length can be tested in compression; an Emery horizontal machine with a load capacity of 2 300 000 lb. in compression and 1 150 000 lb. in tension for lengths up to 33 ft.; and several smaller machines. These machines are now in laboratories at Washington. To meet the demand for tests at higher loads, the Bureau would like to be able to acquire a machine of the Emery type with a capacity of 20 000 000 lb., and tentative plans for such a machine have been prepared.

Tests of Columns.—One of the most important investigations on columns conducted at the U. S. Bureau of Standards was made under the jurisdiction of the Special Committee on Steel Columns and Struts of the Society.* This investigation, which consisted of a series of tests of 180 columns of various sizes, design, and material, has been very thoroughly discussed, although it is

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1583.

possible that the data available from these tests have not been utilized to their fullest extent. The program was much more elaborate than was actually completed, and in its Final Report the Committee expressed disappointment at the reception of the report by the Profession.

It might be wise to extend the work of this Committee, and in that connection the speaker wishes to suggest that if such a committee on column tests is again organized that it include, as in the Joint Committee for the Investigation of Sulphur and Phosphorus in Steel (see Section 2), representatives from all interested groups, such as structural engineers, steel and structural manufacturers and consumers, independent metallurgists, best accredited through technical organizations and the Government laboratories, by which procedure it seems certain that the various aspects of the problem will receive due consideration. It is not a reflection on a committee of civil engineers to state that some of the metallurgical and manufacturing aspects of their problem were not adequately considered in planning these column tests.

Another series of tests on forty-two columns was made for the American Railway Engineering Association, the results of which have been published by that Association.*

A third series was made of eighteen large bridge columns described by J. H. Griffith, M. Am. Soc. C. E., and Mr. J. G. Bragg,† including items for the St. Louis Municipal Bridge, the Chicago, Burlington and Quincy Bridge, at Metropolis, Ill., and the Memphis Bridge. It is perhaps worthy of remark that some of these tests were the first and only tests that have been made of alloy steel columns of large size. These alloy steels included nickel, Mayari, and silicon steels.

At the request of the American Bridge Company a comparison was undertaken of solid columns of various thicknesses with corresponding fabricated columns, a total of thirty-nine columns. This is being supplemented with tests now in progress on material furnished by the Bethlehem Steel Company. The latter program comprises seventy-two columns, submitted with the object of rounding out more conclusively the comparison of the behavior of the solid and fabricated column types. Several auxiliary matters are also being considered, such as the effect of variation in size and location of test coupons, comparison of thin and thick sections, and the effect of change in yield point of material on test results of columns. It is hoped to be able to complete and publish this series of tests soon.

Structural Steel Angles.—The U. S. Bureau of Standards has recently published,‡ from material furnished by the American Bridge Company, a series of compression tests on 170 standard rolled structural steel angles, of various design, made from low-carbon steel. The specimen which sustained the greatest unit load for a given slenderness ratio and method of fastening, in most cases, suffered the least lateral deflection, and the angle which bent most, sustained the lowest unit load at failure, the deflection being measured at four-ninths of the theoretical maximum load.

* *Bulletin 21* (No. 223), pp. 474–501, January, 1920.
† *Technologic Paper 184*, U. S. Bureau of Standards.
‡ *Technologic Paper 218*, U. S. Bureau of Standards.

For large slenderness ratios, the average values are well represented by Euler's formula for long columns, calculated for different values of the end fixation factor.

The Karman curves, re-calculated for a yield point of 37 000 lb. per sq. in. and a modulus of elasticity of 30 000 000 lb. per sq. in., represent the average results for small slenderness ratios for several methods of end fixation, except for $\frac{l}{r} = 80$ to 85, where the effect of eccentricity was greatest.

For angles with ends folded, the column formulas considered do not represent the results found in this series of tests.

Short Column Sections.—In the cellular construction of the towers of the Delaware River Suspension Bridge, the speaker understands that there were questions relating to the best distribution of metal in the several parts of the I-sections to be used, that required further experimental study. Accordingly, further I-shaped column sections of silicon steel, 10 ft. in length, of constant depth, and 35½ in., back to back of angles, have been prepared for test purposes by the Bethlehem Steel Company for the Delaware River Bridge Joint Commission, and such tests are now under way at the Bureau.

The flanges of these columns are similar throughout, being 6 by 4-in. by ⅝-in. angles, the variable element being the web thickness and its concomitant variation of ratio of web thickness to unsupported transverse width. The web thicknesses vary from ⅜ in. to 1½ in., giving a ratio of thickness to unsupported width of 61 to 15, the total areas of the sections varying from 40 to 80 sq. in. The four heavier sections have webs composed of two ½-in., or two ⅝-in. plates, stitch riveted. The details of the methods of the tests have been briefly summarized and published.* About one-half the tests have been completed, and it is hoped that the results of this investigation will be available in the near future.

Inferences from Column Tests.—The major column investigations at the U. S. Bureau of Standards, in addition to furnishing much valuable information about the individual columns under test, have indicated the possibility of one general conclusion of great importance. Within the slenderness range of ordinary commercial construction, say, from values of $\frac{l}{r}$ between 30 and 90, the major factor in determining the strength of sturdy columns is the physical character of the material of which they are constructed and, in particular, the behavior of this material near the yield point stress. As a striking example of this fact may be mentioned two columns, of identical construction, tested in the same manner in the same machine. One carried a load of about 3 500 000 lb. and the other only about 2 400 000 lb. Coupons cut from the two columns showed that the yield point of the steel in them was nearly in the same ratio as these loads.

Dr. Tuckerman who is working on this problem at the Bureau of Standards, considers that the general laws of column action, therefore, can only be worked out when the relationship between column strength and physical

* *Engineering News-Record*, Vol. 89, p. 986, December 7, 1922.

properties of the material have been thoroughly studied. A sufficient theoretical foundation seems to have been laid in the work of Engesser, Considére, Jasinsky, Meyer, Karman, Southwell, and others, but there are very few experimental investigations in which the results of column tests are correlated with thorough coupon tests of the material. The preliminary experimental work in this field must almost of necessity be made on small specimens, as the technical difficulties of producing large columns with a fine gradation of physical properties are great and the cost would be excessive.

The Bureau has in progress an investigation of this kind covering small specimens of materials of widely differing yield points and different moduli of elasticity. The results of this series of tests should help to make clear the way for a better understanding of the general laws of column action.

In addition to this study of general laws of column action, there is a wide field of usefulness in particular tests of particular structures. In this field, because of its equipment of large testing machines, the U. S. Bureau of Standards can perform a very useful service. As an illustration of this type of research, the testing of 143 duralumin girders for the new Navy dirigible, *Z-R1,* has already been mentioned. The work at present being conducted for the Delaware River Bridge Joint Commission is another illustration. These tests will enable the engineers to proceed with the construction of the towers with an added feeling of security in the design.

There evidently is much work of this kind of immediate value to structural engineers, which could profitably be done at the U. S. Bureau of Standards, and, in the speaker's opinion, the main problems of this nature should be attacked with the co-operation of a representative committee, as outlined previously.

6.—New Testing Devices

The speaker wishes to call attention to two new testing devices recently developed at the U. S. Bureau of Standards. These improvements, one of which is a method of visual examination of stresses in metal structures as the yield point is attained and the other an instrument for measuring variations in strains or stresses of materials or structures under load, give promise of great usefulness in engineering.

Method of Mr. R. S. Johnston for Detecting Lüder Lines.—In the early stages of the investigation of the members for the Delaware Bridge, it was apparent that the so-called Lüder lines which appear under certain conditions, after the material has been stressed to the yield point, would offer valuable data on the distribution of stress in the test specimen. A study of this question led to the development of a simple method of making these Lüder lines readily observable, which permits of photographic recording of the development of stress. The method consists in applying a thin wash of Portland cement.

It is believed that this development at the Bureau is the first time visualized stress distribution has been exhibited so clearly on other than laboratory test specimens, and being adaptable to such large test members as

TABLE 6.—PUBLICATIONS OF THE U. S. BUREAU OF STANDARDS RELATING TO STRUCTURAL STEELS.

Title.	Author.	Class of paper and number.	Number of pages.	Date.
Melting Points of the Iron Group Elements by a New Radiation Method	G. K. Burgess	Sci., 62	11	Apr. 5, 1907
Critical Ranges A_2 and A_3 of Pure Iron	G. K. Burgess and J. J. Crowe.	Sci., 213	56	Sept. 22, 1913
Correlation of the Magnetic and Mechanical Properties of Steel	Charles W. Burrows	Sci., 272	38	Mar. 29, 1916
Notes on the Microstructure of Iron and Mild Steel at High Temperatures	Henry S. Rawdon and Howard Scott	Sci., 356	9	Mar. 15, 1920
The Strength of Reinforced Concrete Beams, Results of Tests of 333 Beams (First Series)	Richard L. Humphrey and Louis H. Losse	Tech., 2	200	June 27, 1911
Surface Insulation of Pipes as a Means of Preventing Electrolysis	Burton McCollum and O. S. Peters	Tech., 15	44	Jan. 5, 1914
The Determination of Phosphorus in Steels Containing Vanadium	J. R. Cain and F. H. Tucker	Tech., 24	11	May 17, 1913
Earth Resistance and Its Relation to Electrolysis of Underground Structures	Burton McCollum and K. H. Logan	Tech., 26	48	Dec. 20, 1915
Temperature Measurements in Bessemer and Open-Hearth Practice	G. K. Burgess	Tech., 91	20	May 8, 1917
Effect of Temperature, Deformation, and Rate of Loading on the Tensile Properties of Low-Carbon Steel Below the Thermal Critical Range	H. J. French	Tech., 219	52	Aug. 22, 1922
Preparation and Properties of Pure Iron Alloys; 1. Effect of Carbon and Manganese on the Mechanical Properties of Pure Iron	Robert P. Neville and J. R. Cain	Tech., 453	33	Oct. 16, 1922
Result of Some Compression Tests of Structural Steel Angles	A. H. Stang and L. R Strickenberg	Tech., 218	17	Aug. 3, 1922
Effect of Heat-Treatment on the Mechanical Properties of One Per Cent. Carbon Steel	H. J. French and W. G. Johnson.	Tech., 206	31	Dec. 27, 1921
Manufacture and Properties of Steel Plates Containing Zirconium and Other Elements	G. K. Burgess and Raymond W. Woodward.	Tech., 207	54	Feb. 1, 1922
Thermal Stresses in Chilled Iron Car Wheels	G. K. Burgess and R. W. Woodward	Tech., 209	34	Mar. 18, 1922
Standard Gauge for Sheet and Plate Iron and Steel		Circular 18	....	
Metallographic Testing		Circular 42	....	
The Testing of Materials		Circular 45	....	
Invar and Related Nickel Steels		Circular 58	....	
Some Unusual Features in the Microstructure of Wrought Iron	Henry S. Rawdon	Tech., 97	25	Sept. 20, 1917
Observations on Finishing Temperatures and Properties of Rails	G. K. Burgess, J. J. Browne, H. S. Rawdon, and R. G. Waltenberg	Tech., 38	63	Apr. 28, 1914
Tests of Large Bridge Columns	J. H. Griffith and J. G. Bragg	Tech., 101	139	June 27, 1918
Tests of Eighteen Concrete Columns Reinforced with Cast Iron	John Tucker, Jr. and J. G. Bragg	Tech., 122	....	
Metallographic Features Revealed by the Deep Etching of Steel	H. S. Rawdon and Samuel Epstein	Tech., 156	24	Mar. 19, 1920
Tests of Bond Resistance Between Concrete and Steel	W. A. Slater, F. E. Richart and G. G. Scofield	Tech., 173	66	Nov. 1, 1920
Five Tests of Building Columns	S. H. Ingberg, H. K. Griffin, W. C. Robinson and R. E. Wilson	Tech., 184	375	Apr. 21, 1922
Influence of Phosphorus upon the Microstructure and Hardness of Low-Carbon, Open-Hearth Steels	Edward C. Groesbeck	Tech., 203	33	Nov. 21, 1921
Tensile Properties of Some Structual Alloy Steels at High Temperatures	H. J. French	Tech., 205	18	Dec. 20, 1921
Physical Properties of Materials: I—Strengths and Related Properties of Metals and Certain Other Engineering Materials		Circular 101	....	
The Structure and Related Properties of Metals		Circular 113	....	

are here discussed, seems to offer a new and interesting field in structural engineering research.

Electric Telemeter.—The development of a carbon-resistance type, remote reading and remote recording device for the measurement of stresses, strains, and forces, has been brought to a point where satisfactory practical application in field and laboratory work is being made. The operation of the device depends on the fact that a stack of carbon disks, when subjected to pressure, undergoes a change of length as well as a change of electrical resistance, and means have been devised to make these changes reproducible with sufficient accuracy for many engineering instruments. The apparatus has been used for making photographic records of stresses in the stay cables in aeroplanes during flight, of strains in steel bridge members while trains were passing, and for various laboratory purposes, including the testing of structural members, such as duralumin girders for the Navy Department and cellular members of the Delaware River Bridge towers. It has the advantage that simultaneous records can be made of the strains in different parts of a structure, the recording apparatus being placed at a convenient point while the elements that are attached to the members are placed as desired, the necessary electrical connection between them being made by three conductor leads which may be several hundred feet in length if necessary. A forthcoming *Technologic Paper* by Messrs. B. McCollum and O. S. Peters will describe the apparatus and its applications in detail.

7.—Conclusions

The speaker has not attempted to discuss exhaustively the subject of steel for engineering structures. The object has been twofold: First, to emphasize somewhat the metallurgical features of some of the broader aspects of the problem and the importance of associating the metallurgist with the engineer in the solution of problems dealing with steel for structures; and, second, to outline briefly a statement concerning some of the investigations in this field recently completed or in progress at the U. S. Bureau of Standards. An opportunity has not been given to verify all the historical statements and they are set forth with reserve. In Table 6 is given a list of publications of the Bureau of Standards relating to structural steels.

In the short time available for the preparation of this paper, the speaker has had the co-operation of several of his colleagues, as mentioned previously. He is also indebted to Messrs. P. D. Merica and H. T. Morris for information on alloy steels, and to Messrs. W. Spraragen and F. P. McKibben for certain facts relating to welding.

DISCUSSION

CLEMENS HERSCHEL,* PAST-PRESIDENT, AM. SOC. C. E.—The speaker has considered himself, in some respects, a little out of tune with some parts of what is called research; and, therefore, the following ideas are stated diffidently and for consideration. He has always been guided in his work by aiming at engineering utility.

There is a distinction between research merely for the sake of research and engineering research, so much so that the speaker fears there has been a misuse of the word, "research". There is a difference between making experiments of a kind that may be likened to amusing oneself, and an engineer's work in that line, which aims at a definite object. This may be illustrated by the work of Pasteur, who was recently commemorated on the occasion of the 100th anniversary of his birth. Pasteur was a man whose life work was far removed from engineering, and although called a scientist, and essentially living the life of a scientist, with his whole career devoted to science, he never did a thing that had not a definite, useful object for its aim. At one time, it was the cure of rabies; at another time, it was the prevention and cure of anthrax, in order to save the lives of sheep; or, again, it was the removal of a pestilential parasite from the vineyards of France, as well as a great many other such kinds of work. For years, his aim invariably was toward a definite object, and one that was useful to mankind.

It has always seemed to the speaker that every investigator, every one working in the so-called line of research, should criticize himself or question himself, and in this way endeavor to follow aims of the kind referred to. The speaker does not propose to sit in judgment on the work of those who are acting as investigators; he wants them to do it themselves to themselves, to feel that what they do is for the purpose of advancing the cause of mankind, not merely to find out something new and record it in a scientific paper or in a book. That kind of work was useful in its time; but it has been multiplied; it has increased to such an extent, and there is so much buried in those books and in the transactions of so-called learned societies that if a man wants certain knowledge to-day, it is just as easy, or easier, to go out in Nature and by experiment, seek, and thus get it, as it is to search for it in the libraries.

What the speaker has said tends to distinguish the work of engineers from what is mellifluously called research in "pure science" and is claimed as the only original source of advance in the arts and sciences. He is questioning the validity of this claim. It may surprise many to be told (it is not the commonly accepted or popular creed, indeed it goes against it) that useful inventions frequently, perhaps generally, precede their scientific explanation, or alleged foundation. For instance, the speaker has doubts whether Marconi reasoned from Hertzian waves to wireless telegraphy. It is much more likely that he experimented with electrical apparatus, succeeded in sending messages through the air, and, later, had all this explained and computed by a con-

* Hydr. Engr., New York City.

sideration of Hertzian waves. It is certain that all the beneficent inventions of Edison were made without a knowledge of "pure science".

The engineers of America have had a great opportunity for the advancement of engineering knowledge afforded them by the foresight and generosity of an eminent member of their profession, when he created Engineering Foundation. Let it not disappoint the cherished hopes of the engineers of the country, by a perversion of its funds, in doing work other than that of engineering. The scientists of the country have had their particular work amply endowed. Some years ago one of them, in charge of a huge endowment of this kind, laid down the rule and criterion that the funds of which he had charge should not be used for anything that had a useful end for its aim and object. Engineers should be prepared to accept his challenge and are already stating that Engineering Foundation should have for its rule and criterion the opposite view, and should support nothing that has not some object that is directly intended to be useful to mankind. Thus will be maintained the engineer's desire to be of service, rather than to follow only a more or less selfish quest of pleasurable emotions.

William G. Atwood,* M. Am. Soc. C. E.—The various research projects have been outlined quite fully by Mr. Flinn, but it may be possible to add a little information concerning the work of the Committee on Marine Piling Investigations. This Committee was organized as a result of the recommendations of the San Francisco Committee which was formed on account of the total destruction by marine borers of many harbor structures in Upper San Francisco Bay during 1919 and 1920. The San Francisco Committee found that the field of investigation could not be covered adequately by a local committee and, to be successful, the work must be done on a National scale. The problem was to devise means for protecting wooden structures from marine borers and to find the causes and remedy for the deterioration of concrete in and near sea water.

The most urgent phase of the problem seemed to be a study of marine borers, and, therefore, this was the first to be investigated. It was found that there was little knowledge of the distribution of the various species of borers or of their capacity for destruction. In order to obtain accurate information along these lines, test boards were placed and are being maintained by the railroads, various Government bureaus, municipal bodies, and industries owning property on salt water. The first of the boards was placed in June, 1922, and the last one early in 1923, and they have already yielded so much information that within a short time it will be possible to discontinue a number of them. Many new species of borers have been discovered, some of them with great power of destruction, and some previously known species have been found to have a capacity for living under conditions which had been thought would prevent their existence.

Good creosoting has long been recognized as the best method of protecting timber, but, in some localities, even well creosoted timber seems to have com-

* Director, Committee on Marine Piling Investigations, National Research Council, New York City.

paratively short life. The U. S. Forest Products Laboratory and the Chemical Department of the University of California are both making chemical studies of creosote and methods of impregnation, hoping thereby to enable the quality of the oil to be improved and the impregnation to be made more thorough. The University of California is also experimenting with several metallic salts which are locked in the wood by a process developed in the laboratory.

The Assistant Secretary of the U. S. Department of Commerce, by designation of the President, has become a member of the Committee, and arrangements have been made for a study, under the direction of the Chemical Warfare Service, of an entirely new series of poisons which may be injected into the timber. The Chemical Warfare Service will also endeavor to develop a method of poisoning the water somewhat in the same general manner that a gas barrage is placed on land. If this process can be developed, it will be possible to protect existing structures built of untreated timber when they are threatened by attack, and it may also make it economical to build structures of unprotected timber in places where borers are known to exist. Some of the methods for poisoning the water suggested will be practicable and inexpensive, if their efficiency is demonstrated by the proposed tests. The cost of this study, about $20 000 per year, will be paid by allocations from the funds of the various Federal Bureaus owning and maintaining structures in sea water.

The deterioration of concrete is as great a source of loss to property owners as the marine-borer damage to wood—if not greater—and because the use of concrete is increasing, the prevention of the deterioration is becoming of greater and greater importance. The records collected by the Committee indicate that there are few concrete structures in salt water, which do not show deterioration after a life of ten to fifteen years. Many of them show serious deterioration in even less time. The causes for this action are not known, but it appears that methods and materials that will give good results in fresh water, will not do so in salt water.

It seems probable that the use of unsuitable aggregates, too much or too little gauging water, and improper mixing and handling, have been partly responsible for many failures, but careful study shows that chemical changes in the concrete are probably the principal causes of deterioration. These chemical changes may be greatly accelerated by the use of unsuitable construction methods or aggregates. Certain investigations of the Geo-physical Laboratory of the Carnegie Institution and of the U. S. Bureau of Standards seem to indicate the possibility of improving the cement itself. The Committee is planning a series of experiments based on the information gained from existing structures and previous laboratory studies, with the hope of finding the real causes of failure. After the causes are found, the remedies should easily be developed.

Both phases of the investigation are being based on the best scientific advice obtainable, as it is the feeling of the Committee that the solution of both must be based on pure science. For that reason, the membership of the Committee includes chemists and biologists as well as engineers, with the

idea that only by the co-operation of pure science and applied science, represented by the engineers, a satisfactory solution can be found.

The Committee has had the enthusiastic support of the American Railway Engineering Association and the financial backing of the railroads. The Government support is coming largely from the Navy and Army Departments and from the Department of Commerce. The work thus far accomplished, has cost about $25 000 in cash and probably twice that amount in contributed services. During 1923, the cash expenditure will probably be $50 000 to $60 000, including the Government appropriations, and the contributed services will amount to at least one-third more than the cash expenditures.

The first preliminary report of the Committee covering only a part of the marine-borer phase of the study will appear in a *Bulletin* of the American Railway Engineering Association. Further reports will be published as rapidly as the information developed will permit, so that engineers who are designing and building structures in salt water may have the latest information available.

H. H. Rousseau,* M. Am. Soc. C. E.—The speaker is in entire sympathy with Mr. Atwood and Mr. Herschel on this subject. From engineering research come progress and growth, and any means of furthering the ends of engineering science by promoting engineering research should hold a high place in the mind and attention of every engineer. The day has passed when civilization should be obliged to trust to the supplying of its daily increasing needs through the research work of individuals working separately, each with more or less limited knowledge, resources, and facilities. This is the age of specialization and organization; it is very fitting, and accords with present-day thought and practice, that united and co-operative effort should be directed toward increasing the production of new ideas and processes in the practice of engineering, and accelerating the rate of development in all lines of engineering science—all for the betterment and advancement of the human race.

It is often claimed that military science and the implements of warfare develop during war at a rate one hundred times faster than during peace. Is not this a striking example of what can be accomplished by intensive, co-operative, research work under the urge of necessity? And who can say that mankind itself is not confronted to-day with a contest with Nature of an urgency akin to an actual state of warfare, the outcome of which, ultimately, may possibly determine man's very existence? Every day sees a diminution of our natural resources, many of which can never be replenished, and most of the activities of our every-day life are accompanied by glaring wastes that might be avoided. On every side we are aware of operations being performed at efficiencies that should be many times greater. The necessity for conservation and economy, by the human race as a whole, and for the adaptation and use of new methods and materials, is growing constantly greater. Should not the problem of providing for the manifold requirements of an ever-increasing population with diminishing resources be a spur as effective as a state of actual warfare to a realization of the necessity of a permanent campaign for

* Rear-Admiral (C. E. C.), U. S. N., Washington, D. C.

carrying on intensive, organized, engineering research work in all its branches? And may not the results of such a campaign, directed by intelligent effort, and availing itself of 100% of the united ability, energy, knowledge, and resources of the engineers of the country, be similarly a hundred-fold greater than any progress that could be made by engineers working merely as individuals? No one would claim in war that soldiers fighting as individuals, even though much superior in numbers and equipment, could hope for anything like the measure of success that comes only through organization and discipline. Nature is jealous of her secrets, and organization and a well-directed plan of campaign are necessary to attain victory over her.

The Navy Department has been very glad to assist with all its resources the work of Engineering Foundation in investigating the attacks of the *teredo* on piling and timber, and in determining the best methods of securing immunity therefrom, as well as the deterioration of concrete by the action of sea water. Both these subjects are of much interest, and any solution of these problems will be of great value to the Government.

W. C. Cushing,* M. Am. Soc. C. E.—Railroad engineers are much interested in almost any line of engineering research, and as far as the speaker's connection with the Committee on Research is concerned, he does not know of any proposed research work that is not needed by the Engineering Profession. Such researches are not being conducted for the purpose of furnishing interesting study, but for the ultimate practical use which can be made of the work. The speaker, therefore, does not altogether understand the distinction between the two lines, that has been mentioned, and he certainly hopes that the opportunities for this kind of work will increase.

Charles Rufus Harte,† M. Am. Soc. C. E.—Engineers, particularly those who are in active work, are too apt to forget that what are called the laws of Nature are merely statements of what has been observed under certain circumstances, and though such laws may state exactly what occurs, they exist because of the occurrence; the occurrence does not follow because of the existence of the so-called laws, and, in many instances, owing to the fact that the observations have not covered all the factors, the so-called "laws" are themselves incomplete or inaccurate.

As any line of investigation, whether it is investigation in the sense used in the laboratory or is what is no less investigation, develops the continued production of something, there are found at all stages occurrences of various kinds which often seem to have no particular relation, or certainly no important relation, to the main work, but as the development is carried on, it almost invariably results that new troubles or peculiarities are encountered, the explanation of which, in part or in whole, is found in this apparently unimportant and previously neglected occurrence.

Nothing illustrates this more clearly than the development of radio. The curiosity of Thomas Edison compelled him to ask why there should have

* Engr., Standards, The Pennsylvania System, Philadelphia, Pa.
† Constr. Engr., The Connecticut Co., New Haven, Conn.

occurred such an apparently inconsequential thing as the blackening of the tube of an incandescent lamp, not uniformly but locally; and it was the investigation of this which, to his mind accustomed to see in everything an indication of an underlying cause, resulted in the development of the thermionic bulb and all that depends on it.

Engineers should rid themselves of any mistaken idea that research is merely the closet pastime of the "theoretical college professor", using this expression in the mistaken sense that is attached so often to the word "theoretical". Research is "finding out", and it is of the utmost importance to the working engineer that every assistance be given to research work because, on the one hand, by the study of difficulties it serves to find the answer, and, on the other, by the study of peculiarities which are not at once referable to known facts, it serves to broaden knowledge, both in the working and the purely intellectual fields.

F. E. SCHMITT,* M. AM. SOC. C. E.—Professor Talbot and Mr. Flinn seem to have covered the subject very fully. One little difficulty, however, might well be examined and borne in mind. The essence of research, as the speaker sees it, is the discovery of something that we did not know before. On the other hand, engineering work is essentially a matter of working with tools that we have and that we consider satisfactory, and the common attitude of the practicing engineer toward his work and his problems is that the means at hand are ample—he is bound to take that view, or he cannot have confidence in the results of his work. The beginning of research is doubt as to whether the available means are ample. It is difficult for engineers—speaking primarily of civil engineers—to put themselves into that doubting frame of mind, and the speaker thinks that we must charge to this phase of the matter—in part at least—the difficulty of collecting the problems which face every engineer in his work, and of bringing them to a focus so that research can begin.

One associated point may be worthy of mention: Mass research is, in a certain measure, applicable to civil engineering problems; but it has always seemed to the speaker that the majority of these problems relate to matters that individually are easily investigated. All over the United States we have many laboratories and many keen and enthusiastic men who, if they can be put into contact with worth-while problems, will set to work to solve them. Research, however, is discovery. One cannot hire it, but must depend for it on spontaneous activity; and so the problem of stimulating a spirit of research in the Society, and thereby galvanizing its membership into a desire to advance the scientific knowledge of the Profession, depends on enlisting the individual effort of members all over the country, on stimulating a wish to join in the work of discovering new things and to put their individual abilities and powers into the general service. For that, again, it is necessary, first, to determine the individual questions calling for research, and then to bring them to the born investigators, those who have the capacity and the facilities. In this direction little has yet been done; and not much can be

* Associate Editor, *Engineering News-Record*, New York City.

done unless the members generally appreciate the importance of this work and give it their co-operation.

HENRY GOLDMARK,* M. AM. SOC. C. E.—Relative to the admirable paper by Mr. Burgess, on the work that has been done and is being done by the U. S. Bureau of Standards, the speaker recently had occasion to witness some of that work, particularly the tests in welding, and also the large number of tests on the duralumin columns used in the airship, *Z R-1*, now under construction by the U. S. Navy Department. The Bureau of Standards is not sufficiently supported; it is cramped by a lack of clerical assistance, and its work is limited by a lack of funds. Just what the Society as a whole, or its members individually, can do to induce Congress to make larger appropriations, the speaker does not know, but the members should use every opportunity afforded them to educate public opinion to the necessity of research activities, particularly those which are so admirably conducted by the U. S. Bureau of Standards.

W. A. SLATER,† M. AM. SOC. C. E.—The speaker is glad to know that the U. S Bureau of Standards has been able to help in some of the work of engineering research, and that the U. S. Department of Commerce has been able to contribute in a substantial way.

If engineers would make more use of some of the Government institutions for research, the speaker believes that it would make those institutions much more useful than they are at present. Engineers and industries would make the Bureau of Standards, for instance, feel that it was needed, if they would bring pressure from the outside to get done that which is needed, and the Bureau would be able to do this work much more effectively.

A great deal of attention has recently been given in the search for laws governing the strength of concrete. The problem has been studied from many angles; the search has been made in as great detail as the facilities available in the different laboratories have permitted. Some investigations have been made, which might appear like a misdirection of effort, but all the serious work that has been done is contributing toward the solution of the problem. All the laws have not been established, but it seems that there has been very little really wasteful research on the subject of the strength of concrete.

Another phase of research that has not been discussed is that of making available in English the literature in foreign languages. The speaker knows of no organization in the United States that is doing such work. Foreign technical papers discuss in more detail the results of research than American technical papers. No technical paper in this country seems to feel that it can afford the space or the expensive work that a great deal of this kind of publication would involve; and the speaker would inquire whether there is any place for such a publication in the work of Engineering Foundation, or any of the affiliated organizations that have been mentioned. A number of valuable researches published in the German language, for instance, could be cited, that are not available in English, and the speaker believes that their translation would assist greatly in avoiding duplication in the United States of

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† Urbana, Ill.

work already done abroad, in helping to interpret work done in this country, and in furnishing data that otherwise may never be obtained.

ALFRED D. FLINN,* M. AM. SOC. C. E. (by letter).—Engineering research is so comprehensive a subject that much relating to it might be written. In closing this discussion, a few words should be added about Engineering Foundation. In its few years, the Foundation has been an instrumentality for accomplishing certain notable things, mentioned in the paper. It has recently commenced a new era in its development. Having aided substantially in establishing National Research Council, and, particularly, the Division of Engineering, it has discontinued its financial contributions, in order that its resources may be devoted more directly to the researches and investigations of its Founder Societies. It continues cordial relations with all the divisions of National Research Council.

Important facts to bear in mind concerning Engineering Foundation are that it is the joint instrumentality of the Founder Societies for research and investigation, and that, no matter how great its funds may become, its greatest asset will always be the mental resources of technical men associated in these great National engineering societies.

The term, Engineering Foundation, has three meanings:

First.—A body of educated men associated with the intent of fostering researches and investigations to apply "for the good of mankind" the sciences related to engineering.

Second.—An endowment fund held in perpetuity by United Engineering Society, as trustee for the Founder Societies.

Third.—The department of United Engineering Society based on the mental resources of the Founder Societies and the endowment fund, comprising men, means, and activities.

Engineering Foundation is a body representative of the great National engineering societies as contrasted with the usual self-perpetuating board placed in charge of a great fund for public purposes. It serves the Founder Societies in numerous ways, and is one of their joint instrumentalities for advancing the Profession of Engineering and promoting the welfare of the Nation.

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CITY PLANNING

TECHNICAL PAPERS
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REGIONAL PLANNING

By Nelson P. Lewis,* M. Am. Soc. C. E.

Early attempts at city planning were very limited in scope, having been confined almost entirely to projects for civic centers, many of which were ambitious, but few, if any, ever were completed. Subsequently, plans were made for particular and restricted areas of cities, most of them having been for new suburban real estate developments. In very few cases, even yet, have plans been made for the more orderly development of towns or cities as a whole, including an effort to correct some of the obvious mistakes due to the haphazard growth of the older built-up portions, as well as those where development could still be directed along more rational lines, with better adaptation to topography.

Most of these plans have been the results of efforts of groups of private citizens, or of civic or commercial organizations, and, lacking formal adoption by competent municipal authorities, they have not been translated into official and binding plans. Many planning commissions and boards have been created, some to be starved by inadequate appropriations, some to be abolished by the local authorities which created them. Others are functioning and rendering valuable service, especially in controlling the character of new developments, both within and without the city limits. Emphasis should be placed on the words, "without the city limits", as indicating a new and growing realization, in many instances taking the form of statutes, of the fact that it is useless to talk about correcting defects in the existing city if this same lack of intelligent planning is allowed to proceed in the territory which will soon become part of the city. Many cities, in their special charters, or by general city law, have been given the power to control the subdivision of property not only within, but for some miles beyond, their corporate limits.

Even the most intelligent planning has been predicated on the idea that the city is to extend outward and ever outward, that it will continue to absorb one suburb after another, or if it cannot annex and assimilate them, that it will spread over the unoccupied land between it and them. The result of the decennial enumeration is awaited with eagerness and if it does not meet the expectations and gratify the ambition of the town for bigness, a recount is demanded. This unbroken expansion, although jealously maintaining the old centers, results in the most serious difficulties in supplying transit within the city and in accommodating those whose occupation requires them daily to enter and leave the business district. The transit system is likely, as in New York City, to be developed in such a manner as to aggravate these conditions still further, so that the concentration of population in all large cities is getting to be a serious menace, and the problem is fast becoming one of decentralization.

How are the cities trying to cure the conditions resulting from the unwholesomely rapid growth of the past few decades, a growth for which they

* Cons. Engr., New York City.

were not prepared and which has brought with it great congestion in housing and traffic, so that in the homes, especially those of the poor, conditions are unfavorable to health and morals, and the use of the streets involves serious risk of physical injury? Scarcely a week passes without some new proposal for relieving not only existing congestion, but also for meeting that which will come with further increase in numbers, accompanied, doubtless, by further development of nervous excitability and further decline in health and morals. New streets, over or beneath those existing, or even cut through intensively built blocks at enormous cost, are proposed to care for existing and future traffic. New rapid transit lines are urged, located so as to bring more and more people to existing centers of finance, business, and amusement. These proposed changes are doubtless necessary to meet or to cure existing conditions, but the arguments in their favor nearly always refer, with more or less pride, to the increase in population, in business, and in wealth, which future years will bring. There is almost always an unconcealed conviction of the glory of mere bigness. There is seldom a suggestion of the need of trying to prevent the further development of conditions which render the problems still more difficult of solution. Larger areas must be studied and instead of beginning with present centers and providing for expansion outward, absorbing one after another of outlying towns and villages, it is time to begin at the outer edges of a great metropolitan district, to study existing communities and strive to encourage the development of their centers, to create new social, commercial, and industrial centers, and strive to protect what is worth saving from being drawn into the vortex at the big center and absorbed bodily by it, with no line of demarcation and with no barrier to the steady outward movement of population.

This is what the speaker has in mind when he refers to Regional Planning. The idea is not new. Under quite a different name, the need of such planning was emphasized by Ebenezer Howard at the close of the Nineteenth Century, especially in his book on "Garden Cities of To-morrow". This was soon followed by the establishment of the first "garden city" at Letchworth, England, about thirty miles from London. The idea appealed strongly to the English people who have created a number of new municipal units which, although within a metropolitan territory, are designed to be largely self-contained, not having one industry, but a variety of different industrial establishments, which tends to insure against the serious situation likely to develop in a single-industry town as the result of a bad depression or the cessation of activity in its particular industry. The avowal of such a purpose may cause protest, the kind of protest voiced by certain municipal officers or by local boosters already referred to, who protest the results of the Federal census.

The establishment of such new, self-contained, centers, however, will fail of its purpose unless they are protected not only against their own over-growth, but also against the encroachment of other similar centers or of the great city of which they may be satellites. The British garden cities have insured themselves against such a fate by acquiring rings or zones of wooded or agricultural land which, owned by the town, can be made revenue-producing by

scientific forestry or through leasing or can be devoted to public recreation. The British Town Planning Act, applying to the entire Kingdom and administered by one central authority, affords an excellent medium through which such a policy can be carried out by those towns that wish to adopt it, even though there is a noisy minority in opposition.

Another conspicuous feature and advantage of regional planning is the opportunity that it offers to co-ordinate principal highways or traffic routes so that communication with the big center and between the subordinate centers may be direct and of ample capacity. In developing such a comprehensive system, the time has come when account must be taken of the extraordinary development of the transportation of goods and passengers on public highways, by means of motor trucks and motor stages. This development will force the provision of separate routes, or, at least, of separate roadways, for vehicles of this kind.

The idea of regional or metropolitan planning has been very slow in developing notwithstanding the object lessons which have been afforded by other municipal undertakings in which co-operative action proved to be necessary. Perhaps this is nowhere more forcibly illustrated than in the Metropolitan District of Boston, Mass., where, years ago, what are now nearly two score cities and towns, found it necessary to create Metropolitan Commissions for the development of their water supply, their main drainage, and their park systems. In the case of planning for more orderly growth, however, more convenient means of communication between different parts of the District, and for some means of controlling the use of private property which will be fair to each municipal unit, no machinery has yet been devised.

In New York State, Buffalo, the Tonawandas, Niagara Falls, and other near-by towns are trying to work out a regional plan for the Niagara Frontier District, which will involve even international problems, as the plans must necessarily include connections with the Canadian side of the Niagara River. In California, a Los Angeles County Regional Planning Conference has been organized through which it is hoped that the group of towns in the Los Angeles District and the territory lying between them may be considered as a whole. At the fourth meeting of the Conference, held on September 16, 1922, what were designated as specifications for a regional plan were agreed on, in which some of the fundamentals of such a plan are tersely expressed. Among other things, emphasis was placed on the need of locating regional highways which would by-pass congested business centers so that through travel should not be subjected to the congestion of local business, nor should a large volume of through travel add to local business congestion; there should be a minimum of cross streets along main traffic arteries, long blocks along principal thoroughfares being encouraged; local roads should not cross parkways and boulevards at grade, if this could be avoided; population centers and sub-centers should be connected so as to permit rapid movement of passengers and freight between them; railroad grade crossings should be eliminated as rapidly as economic conditions permit, etc.

By far the most comprehensive project for regional planning which has yet been undertaken is "The Plan of New York and Its Environs", in which it is proposed to include an area of no less than 5 500 sq. miles, located in three States, with a population of approximately 9 000 000, according to the census of 1920. It will include, in addition to the present City of New York, with its five counties, five additional counties and parts of two others in New York State, eight counties and part of a ninth in New Jersey, and a part of one large county in Connecticut. The special difficulty and the delicacy of the problem are largely due to the fact that there are within the area under consideration nearly 400 different political units—States, counties, cities, towns, villages, boroughs, and townships. No one municipal corporation could undertake the preparation of a plan for such an extensive area without arousing jealousies and the suspicion of imperialistic designs on its smaller neighbors. It follows that a project of this magnitude can be undertaken only by an institution or a group of individuals which has no official connection with any of the political units which would be affected. The Trustees of the Russell Sage Foundation, which was established for the express purpose of "the improvement of social and living conditions", concluded that one of the most effective ways in which this object could be promoted would be through the preparation of a comprehensive plan for the future development of the district of which New York City is the center. A Committee on Plan of New York and Its Environs was appointed under the Chairmanship of Mr. Charles D. Norton. At a meeting held in the Engineering Societies Building, at which the first announcement of this plan was made, the justification for ignoring political boundaries in such a study was admirably stated by Mr. Elihu Root, one of the speakers, as follows:

"A city is a growth. It is not the result of political decree or control. You may draw all the lines you please between counties and States, a city is a growth responding to forces not at all political, quite disregarding political lines. It is a growth like that of a crystal responding to forces inherent in the atoms that make it up."

There is probably no precedent for the manner in which the work of this Committee was organized under four separate divisions:

(1) The Physical Survey, which first undertakes to visualize the problem, mapping existing topographical conditions, showing all transportation facilities, public utilities already developed, the density and distribution of population, provisions for recreation, and all local schemes of improvement in progess, or contemplated.

(2) The Economic and Industrial Survey, analyzing the fundamental reasons for the existence of this great center of industry and commerce, with an inquiry into the economic and occupational activities creating populous districts and following population.

(3) A Legal Survey, including the study of existing laws in the several States or their subdivisions, which control or affect a plan for the area.

(4) A Survey of Social and Living Conditions, which would bring out clearly those factors having a direct bearing on human values and making for health and satisfactory surroundings.

The great concentration of population about New York City may be a source of pride to those who like to think and talk of it as the largest city in the world, but it involves many serious physical, economic, and social problems. In studying the past and probable future population, the total area has been divided into three districts, designated as: (1) the Urban District, consisting of the present City of New York, with Hudson County and the City of Newark, in New Jersey, having an area of about 360 sq. miles and a population of 6 700 000; (2) the Suburban District, including all of Nassau and Westchester Counties in New York State, all of Union County, Essex County (outside of Newark), and parts of Passaic and Bergen Counties, in New Jersey, and the eastern part of Fairfield County, Connecticut, having a combined area of about 1 170 sq. miles and a population of 1 360 000; and (3) the remainder of the total area, comprising about 3 970 sq. miles, which is called the Rural District, with a population of 940 000.

The increases in population during the census periods since 1850 have been plotted for each of these districts and the curves projected forward to the end of the Twentieth Century. From them, it appears that at the close of the Twentieth Century the Urban District will contain 20 000 000 people, the Suburban District 13 000 000, and the Rural District 4 000 000. The total population for the entire area will reach 20 000 000 in 1960 and 37 000 000 in 2 000. It is interesting to note that the last named total is equal to that of the present combined population of all the Atlantic Seaboard States from Maine to North Carolina, inclusive. If these forecasts should be verified, the average density of population in the Urban and Suburban Districts combined would be equal to the present density in the Department of the Seine, but it makes a great difference whether such density extends over an area of 1 530 sq. miles or over a restricted area of 185 sq. miles, which is that of the Department in which the City of Paris is located. The speaker hopes that these estimates will prove to be incorrect, but they appear to have been confirmed, at least for the next two decades, by estimates of future population of the present City of New York made by the New York Telephone Company, the careful forecasts of which heretofore have been found to be quite accurate. The Company's estimate for 1930 is slightly less than that shown by the curves referred to, and that for 1940 is slightly greater. Although the uneven distribution of population in the large area is striking, the same holds true, to a remarkable degree, in the present City of New York. The New York Census Committee has called attention to the fact that 22.6% of the city's population lives on 1.6% of its area and that 68.8% lives on 10% of its area.

The purpose of this paper is simply to emphasize the need of regional planning. It would be impossible, within reasonable limits of time and space, to describe the studies thus far made and those proposed in connection with the Physical Survey with which the speaker is more particularly concerned, to say nothing of the other divisions of the work. The general scope of the investigations, and the manner in which the results are portrayed, will be indicated by the following brief list of the maps and plans completed and in progress:

I.—Five maps of the entire area, on a scale of 1 mile = 1 in., showing (*a*) general topographic features; (*b*) arterial highways, existing and proposed; (*c*) present and suggested parks and parkways; (*d*) railroads serving the district; and (*e*) sources of water supply and methods of sewage disposal. These maps are supplemented by twelve sectional maps, on the same scale, showing the principal highways, parks, and other open spaces.

II.—A series of maps and charts indicating the distribution and the relative densities of population throughout the entire area, with forecasts to the end of the Twentieth Century. These maps and charts indicate how unevenly the population is distributed, even within the central area.

III.—Maps showing the principal improved roads; the trunk-line highways, as located, and reduced to a theoretical diagram; the proposed motor-truck routes and the possible stages in their development from or in connection with existing roads; and the traffic density on present highways, the result of counts, showing to scale the maximum number of vehicles per hour.

IV.—Transportation, including city transit: The trunk-line railroads serving the district; the passenger service, as indicated by the number of trains daily; the time zones at intervals of 15 min. from New York terminals; the commuting zones for 5-cent differences in one-way trips at commuting rates, based on 300 round trips per year; and the transit system of New York City and contiguous territory, showing subway and elevated lines, surface railroads, and bus lines.

V.—Park and recreation facilities: The existing public parks and suggested additions; the lakes and reservoirs which under proper sanitary control could be made available for recreation; the canals and waterways which could serve the same purpose; the wooded areas available for wild parks or forest reserves; the areas suitable for public camping grounds; diagrams showing park areas in relation to total areas and to population; and maps showing areas held privately as golf and country clubs in comparison with those free to the public.

VI.—Port and industrial development plans showing existing facilities and projects under consideration; areas zoned for industrial use; maps showing, for each county, the number of power plants and their average capacity.

VII.—Map showing incorporated places, mapped and zoned areas, indicating the extent to which the zoning idea has spread.

VIII.—Maps showing pollution of harbor waters in the central area and the location of sewer outlets and of sewage disposal plants.

IX.—A series of plans and sketches showing possible methods of traffic relief and additional traffic facilities in the most congested areas.

X.—A series of aerial maps and of oblique aeroplane views which portray existing conditions and are an invaluable aid in regional planning work.

An attempt has already been made to indicate the purpose of regional planning, especially where there are great centers of population. This purpose may be briefly summarized as follows: To counteract the tendency to excessive concentration of population and industrial activity in certain spots; to simplify the problem of transit between different parts of the area; to bring facilities

for receiving and shipping goods to all parts of the region; to establish a method of controlling the use of private property, consistently and equitably applied through the entire district, by the adoption of what might be called regional zoning; to locate places of public recreation so that they will be within easy reach of all; and to relieve present congestion which has been aptly described as "the crowding of streets by traffic, the crowding of lots by structures, and the crowding of rooms by people". If these results can be attained, it seems reasonable to predict that general health will be improved, a spirit of neighborliness will be promoted, the nervous strain due to tiresome journeys to work and back, under conditions often indecent, will be abated, children and adults will acquire that wholesome zest coming from closer contact with Nature, political dangers of revolutionary temper and mass action will be lessened, and much of the present economic waste will be avoided.

THE ENGINEER AND CITY PLANNING

By George H. Norton,* M. Am. Soc. C. E.

It has long been accepted that the engineer is essential to the planning of certain phases of community development. For sewerage, drainage, and water supply, his services have been indispensable, even from the earliest developments. As a surveyor he has had influence, at times, in the laying of the general plan of highways, which has directed the future development of a city street system. His position has been essentially that of a hired man assigned for specific purposes, with little differentiation between his highest professional skill and his ability as a surveyor.

In Government surveyed areas, the rectangular system has imposed enduring hardships on many cities, ameliorated, in some fortunate circumstances, by the imposition of diagonals where topographic conditions had induced the aborigines to establish reasonable trails, later followed by the pack train and the pioneer wagon, until they were so well established as highways as to maintain their integrity. In some of the older Eastern cities, devoid of dominating topographic conditions, too often this casual development, through trail and road to street, has been without control by features warranting the devious street plan for a modern city. However, there are excellent examples of sound judgment and foresight in the original plans of some cities. Perhaps, to-day, there is not a proper appreciation of the skill and courage of those who more than a century ago went into the practical wilderness and prepared the ground plans of cities, which have reasonably sufficed until such cities have grown in size far beyond that of any city existing on the American continent at the time of the original platting.

If there had been adequate means for the dissemination of engineering knowledge, the broad ideals of the Washington plan might have been extended beyond the moderated application to Buffalo, N. Y. This influence probably came to Joseph Ellicott who was in charge of the Buffalo improvement, through his brother who was engaged with Pierre L'Enfant in planning the City of Washington. As has been stated, this failure in dissemination of advanced planning was due to the lack of engineering publications and of that personal contact so valuable in this and other similar societies.

Having in mind this unfortunate past condition, it is encumbent on the Profession to review the situation and meet the modern planning problem as it has met others.

Large numbers of the older generation of engineers have been associated at some time, and in some way, with railroad location, construction, or subsequent operation. They developed a high engineering conception of the problems involved and created standards which may guide. Much of this development came after, or perhaps more properly, was the larger element in creating literature and associations for the dissemination and discussion of the problems involved. When these or other engineers have entered municipal service, their

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immediate and pressing duties have been the providing of sewerage, drainage, water supply, and pavements suitable for the existing areas and, perhaps, to look forward to the provision of such for adjacent territory. Assistance is seldom provided beyond that necessary to meet pressing demands, and more seldom is it realized that the field of the engineer should be wider.

The growing sentiment of a few decades ago, that American cities as municipal machines were not at par with other American achievements, led the ever-present reformers to demand changes in organization and administration. After having run the gamut of charters and methods to find that they do not change human nature, there has been a sudden awakening to the fact that municipal structures have simply grown from the demands of commerce, industry, and transportation along lines dictated by the especial needs or convenience of each and without thought of the mutual relations between them. They have grown under the stimulus of varied business impulses, not always harmonious in needs, and without definite plan or guidance. This realization apparently came first, not to the engineer, but rather to the artist and the architect who sensed the lack of harmonious development and the unnecessary clash and ugliness of many elements. Hence, it was only natural that the earlier movements in city planning were more along the line of the "City Beautiful".

The general awakening has disclosed the most necessary things for a well ordered city to be definite knowledge of its actual physical condition and a definite plan for its further development. The ills of the municipal body lie in its deformed and unguided growth rather than in its charter or ordinances, and its beautification should be in ordered arrangement and growth more than in surface embellishments. These fundamentals are fairly well accepted, but the details of development are without accepted standards.

The complex problems of arrangement and co-ordination of the varied and often conflicting interests are so great as to demand the united wisdom of those versed in all their phases. There has been developed, therefore, the usual form of planning commissions, composed of representative citizens whose united intelligence should cover the wide field, and to their assistance has been brought available engineering or other professional advice. Where such technical advisers have not been overshadowed by the so-called practical element, scope for excellent results has been given, the balance of judgment of those representing the complicated interests formulating a policy which would give reasonable results when technically applied.

The alternative method of approach in making the engineer the dominating factor in the development of a plan, with the representatives of varied interests acting as advisers, has had less general application. Results alone can determine the relative advantages and much will depend on the wisdom and judgment of the engineer. He must have the support of the profession not only in sympathy, but more in the development of standards of this comparatively new field. This can be done only through discussion in engineering bodies and dissemination through the technical press.

Although the formulation of a definite plan for the guidance and orderly development of a city may have the highest talent and devotion in its preparation, such a plan will avail but little, unless it can be established.

The successful engineer has had his experience in presenting plans to the chief executive. Usually, the plans have been authorized by the directors who feel assured of the backing of the stockholders. The only questions for consideration are the fitness of the plan and the financial elements. In a city plan, however, there is the element of broad practical politics, not of the partisan kind, but in the truest sense. Municipal governments are most highly representative. The elected representatives in whom the ultimate authority lies may be ever so broad-minded and ready to assist in the development of a plan, but they cannot carry out any comprehensive plan unless it is the will of a decided majority of their constituency.

The problem before the engineer entrusted with any planning project, therefore, is threefold: To prepare a plan which has features such as to appeal to the sound judgment of a large part of the citizens; to co-operate with the ever-present public-spirited citizens who are ready to assist in laying the matter fully before the whole electorate in such manner as to win their approval; and with such public approval, to go before the constituted authorities with the plan and a reasonable program, the most essential features of which may be adopted and carried out by the legislative and executive authorities. With the plan thus established in the minds of the people, there should be no reasonable fear that it, as a whole, will perish. Any live city has enough citizens of public spirit and foresight to see that the matter is brought to successive administrations, with public approval assured.

It is feared that some worthy plans made possible by the endeavor of the few, without general public acceptance, through publicity, may fail of achieving the early results which their intrinsic merit really warrants. Having in mind the essentials to the success of any city plan, the engineer interested in such is brought squarely to face the modern conception of his profession. He must realize that his function is not merely that of the trained investigator and establisher of fundamental facts, but rather that it has the wider limits of the human element. Assured in his own conclusions, he must be able to lay the reasons for such before the public in a manner to show their soundness and be able and willing to meet all objections with the convincing facts at his disposal.

The phases of city planning particularly adapted to engineering analysis are many. The first step in any general plan may be designated as reconnaissance, or the broad survey of the resources and probable line of development of the city and its environs. Such a survey is necessary to the formulation of any comprehensive plan. The whole range of transit and transportation facilities, terminals, harbor development, and many other such questions will intrude themselves into any broad study, and, even if not directly included in the designation, must be given weight.

To-day, the most pressing demand for immediate consideration is apt to be for improvement of streets and the street plan to accommodate the traffic of motor vehicles. The civil engineers of a previous generation developed the

lines of transportation by the construction of railroads along carefully surveyed routes. To-day, although 1 000 000 freight cars may be loaded each week to pass over these lines, 10 000 000 motor vehicles are daily passing over streets and roads that were mostly laid out and determined before the construction of an automobile. There has been developed an entirely new mode of conveyance fitted for travel on highways which were laid out for other vehicles. The flexibility and adaptability of these modern vehicles is such that they may use the rural highways and many urban streets if only a reasonably hard and smooth surface is provided. Such a condition has forced on the whole country a road-building and paving industry and has developed new engineering standards in paving design. Except in the territory adjacent to large cities and on main connecting thoroughfares, the roadway wide enough for vehicles to pass safely provides accommodation for its traffic. Within the cities and on their converging highways, however, the newly developed transportation has been imposed on a street plan not designed for such purpose and bounded and limited by costly land and improvements which make difficult any material alterations. This condition now imposes on the engineer the more serious problem, not to design a thoroughfare system for specific needs, but to adapt the existing facilities to present demands so as to require a minimum of expansive changes.

It is not intended to enter into a discussion of the problems of street design, but rather to outline a few of them as typical of the engineering phases of city planning. No standard, as yet, may be considered established and cannot be until it is perfected through experience and analysis.

Widening Main Thoroughfares vs. Providing Parallels.—Most direct thoroughfares in cities have been occupied by surface traction lines with consequent business development. The location is such as to invite motor traffic with consequent congestion. Although surface traction cars interfere with the easy operation of thousands of motor cars, the latter delay thousands of passengers on the former. Diversion of traffic from such streets may be considered detrimental to the developed business interests. The cost of widening by land acquisition would generally be excessive.

Should these throughfares be widened or should parallels be developed to divide traffic?

Will such diversion injure adjacent business or endanger residence property on the parallels?

If such parallels for vehicular traffic are provided, should they preferably be immediately adjacent to developed main traveled streets or should they be intermediate between such?

Intersecting Streets.—Many cities have abolished at large expense railroad crossings at grade. To-day, at intersections of heavy-traffic streets, more accidents occur and more delays to traffic.

Are we to look forward to grade separation at such important points?

Where we continue to operate at grade, must we continue the alternate "stop-and-go" system or can the rotary, continuous system be developed with reasonable land acquisition?

Can the rotary continuous plan be adopted in preference to the reservoir system for holding the waiting vehicles?

In design of a rotary intersection requiring a material slowing down of traffic, can safety elements be introduced by the modification of grades or vertical irregularities?

Must separate provision be made for passage of long fire apparatus?

Vehicles and the Pedestrian.—What relative weights should be given pedestrian traffic and vehicular traffic when in conflict?

What is the effect on pedestrian traffic of long-radius intersection corners?

Is there a practical limit to the widths of pavements due to difficulty of maintaining parallel traffic lines and danger to pedestrians?

Are two parallel roadways preferable to a single one of their combined width?

Diagonals.—Are diagonals as essential to motor-vehicle traffic as they were formerly thought to be for mixed traffic?

Are not the complicated diagonal crossings often as much of a detriment to traffic as the longer routes of a rectangular system?

Do the advantages of radiating diagonals outweigh their tendency to traffic concentration?

These random problems arising in one phase of city planning are herein noted as illustrative of the character of those details which demand engineering solution. To-day, engineers are becoming alive to them and will solve them as they have solved others.

When the engineer is brought face to face with city planning, he must not only be able to master the technical details, as he has in other lines, but he must realize in the broader way that he serves, not a definite executive, but a complex public. He must have the wider vision clearly before himself and, with it, the courage and ability to impress these visions on the minds of the public. The line of reasoning with which he reaches conclusions must be so clear that it appeals to the average citizen. No plan may stand this test unless it is based on sound engineering fundamentals. No plan can come to execution unless it is understood and appreciated by the public. No plan can die, which has been accepted by the citizens as sound and practicable.

The engineer who enters on city planning must realize that his work is not confined to office detail, but that he must enter that broader field of the man among men. No other field to-day offers a fairer range for the highest development of engineering perspective or the formulation of new standards, and certainly none promises a fuller opportunity for the engineer to take his place as a leader of public thought, qualified to guide the development of those greatest institutions, the cities.

PARKS AND PARKWAYS

By Linn White,* Esq.

The original definition of a park was "a large piece of ground enclosed by the Monarch's grant, or by prescription." The word later came to be applied to grounds about the homes of the nobility—a gentleman's estate. Thus, the earlier conception of a park implied restraint—exclusiveness. At present, particularly in America, it suggests the opposite—the sense of freedom—a heritage of the people.

In the history of the development of the park and the park idea, the course of freedom can be traced. In Paris, France, the city of parks and parkways (boulevards), the ancient walls around the fortified city were leveled to the ground in 1670 and what are now the principal boulevards were formed on their site. The Bastille was literally torn to pieces in 1789 by the people, and in its stead there is now the Place de la Bastille with its "Column of July" marking the beginning of the French Revolution. So it is throughout Europe, in all the countries where the development of freedom can be traced, the evolution of the park from a closed possession of the few to an open place of recreation for all the people, may be noted.

In the United States, where the rights of the people are paramount, the greatest development of the public park would be expected—and this is the case. American landscape engineers may go to the older countries for inspiration, but they do not go there to learn how the parks should be situated convenient to the people who need them most, or how the lines of transportation are best planned to get to the parks, what recreational facilities are most practical, or how community health, morals, and respect for law are affected by the proper administration of parks.

Public parks may naturally and properly be placed in two general divisions, Natural Parks and City Parks, and these, in turn, may be subdivided according to development and use. As custom and the scarceness of park literature have not established a definite park nomenclature, other names may be found for these divisions. One might say Natural Parks and Artificial Parks, but this would be bad, as all successful parks, even formal public gardens, must follow the bent and teachings of Nature. Natural parks have been called Wild Parks or Rural Parks. City Parks, also, may be Natural Parks in the sense that the natural conditions and contours may be preserved, although the park is situated within a city.

The City Park may be developed and adapted to the highest needs of the community from most unpromising beginnings. Barren sand lots, marshy wastes, shallow waters of lakes and rivers, even forbidding city dumps, may be the ground on which the park builder practices his art, in fact, often must be, because he seldom enters the field until the choice has been made to satisfy industrial and residential needs.

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Natural Parks, however, are generally situated where Nature has moulded the landscape and the best that man can do is to preserve what he finds and make it accessible.

The writer will attempt to classify the more obvious sub-divisions of the two general divisions of parks, but this classification is by no means conclusive and, in fact, probably none can be made conclusive, as there are too many common and overlapping characteristics:

Natural Parks:

National Parks, National Monuments, and Battlefield Parks;
State and Interstate Parks; and
Forest Preserves and other Rural Parks.

City Parks:

Large parks within city limits or improved to meet city conditions;
Parkways and Boulevards;
Public Squares and Gardens.
Arboretums.
Zoological Gardens.
Recreation Parks (Park Social Centers).

In this list, the first and last items—National Parks and Recreation Parks—represent two extremes of the development of the park idea, both of which are essentially American.

There are eighteen National Parks, with a total area of 10 739 sq. miles. They extend from Lafayette Park on Mt. Desert Island, off the coast of Maine, to Hot Springs, Ark., and the great Yellowstone Park in Northwestern Wyoming. Rather unfortunately, most of the National Parks are in the West, because most of the suitable territory in the Eastern States had passed into other uses. The first National Park was established in 1832 at Hot Springs, Ark.

At Concord Bridge, the beautiful statue of the Minute Man is standing isolated in restricted and insufficient surroundings. The writer remembers finding along an almost deserted byway in the hills of Massachusetts, with woods on one side and a briar-grown fence on the other, a gray moss-grown stone with crude carving commemorating some tragedy of the earlier Indian wars. Perhaps it is not yet too late to accomplish something better in this historical section of the country.

There are thirty-six National Monuments, such as the Petrified Forest of Arizona and the Muir Woods of California. The Battlefield Parks, such as Gettysburg, Chickamauga, and Vicksburg, are similar in certain characteristics to the National Parks, and are to be distinguished mainly by differences in administration.

In some respects, the Battlefield Parks resemble the improved City Parks in that a large number of monuments and markers are erected and numerous roads built, but, in other respects, the original character and contour of the ground is preserved. Forest Preserves, such as those at Chicago, Ill., and other

Rural or Wild Parks are of a different character and might be classed as City Parks, although situated outside the city limits, except that the general idea is to preserve natural conditions, which seldom can be done to any great extent in truly city parks.

A brief description of the Forest Preserve of Cook County, in which the City of Chicago is situated, should be of interest, as it is one of the largest and most recent enterprises of this kind to be started, although it is by no means complete.

The Forest Preserve District of Cook County was organized by an Act of the Illinois Legislature, and was ratified by popular vote of the County in 1914. The Board of County Commissioners administers the affairs of the Forest Preserve, and funds are provided by county bond issues. The lands are acquired by purchase or condemnation, the area now comprising 24 806 acres, stretching in irregular tracts from the southern to the northern boundaries of the County, a distance of a little more than 60 miles.

As the name would imply, the intention is to take principally wooded tracts, although incidentally some open ground is acquired. Southwest of the city lie the Palos Hills which are an irregular group of hills and ravines, a glacial terminal moraine, well wooded, but not especially good land for agriculture. Here, the largest tract of land, 6 688 acres, has been acquired. Most of the other tracts border on streams like the Desplaines River, and include numerous places of historical interest associated with the early settlement of the country, which are being properly marked and preserved. Other activities of the Forest Preserve are the construction of roads connecting with county highways, re-establishment of old trails, preparation of public golf courses, shelters, preservation of springs, establishment of camp grounds, etc. The Board publishes maps and folders showing motor roads and car lines reaching different parts of the Preserve, and offers special opportunities for the establishment of summer camps. Thousands of city dwellers have availed themselves of these camping facilities, not only for vacations, but for the entire summer, and the enterprise is still in its infancy.

Some of the larger projects of the Forest Preserve District are the establishment of a great arboretum and a zoological garden which will be one of the greatest in the United States.

At Riverside, Ill., about three miles outside the western limits of the City of Chicago, 300 acres of land have been donated by a public-spirited citizen for this purpose. The Chief Engineer of the Forest Preserve has recently returned from a trip of inspection of the principal Zoological Gardens of the old world, and active work of construction will begin soon. It will be the intention to give every animal as nearly as possible its natural environments.

The organization of the Chicago Park System doubtless appears to be quite complicated, as indeed it is in some respects, but it is, nevertheless, a workable plan.

There are three large Park Districts in the city, the South, West, and Lincoln (North), which include about four-fifths of the incorporated area of

the city. Besides these districts, there are fourteen small districts, some of which maintain only one or two neighborhood parks or playgrounds.

There is also a Bureau of Parks, Playgrounds, and Bathing Beaches, under the direct control of the City Council, which directs and administers sixty-five small parks, four public natatoriums, and four bathing beaches. Most of the parks under the jurisdiction of this Bureau are small, and many of them might be classed as squares, as they differ in size from a few acres to a fraction of an acre.

The number and area of the parks and boulevards controlled by the three large systems are as given in Table 1.

TABLE 1.

	South Parks.		West Parks.		Lincoln Park.	
	Number.	Acres.	Number.	Acres.	Number.	Acres.
Large parks	6	1 702.61	4	719.55	1	517.59
Recreation parks	15	330.68	11	60.15	4	24.36
Other small parks	4	67.50	5	39.71	2	8.63
Boulevards	32.98 miles	448.07	32.5 miles	457.9	11.75 miles	92.05
Bathing beaches	3	.....	.....	.....	2	.....
Total area	.....	2 568.86	.....	1 275.31	.....	642.63

All the Park Districts exercise their functions under certain Acts of the State Legislature, either general or special, and are independent of the Mayor and Council.

The organization of the South Park District by special Act of the Legislature is as follows: Five members of the Board control the affairs of the District under the corporate name of the South Park Commissioners. These Commissioners are appointed by the twenty Circuit Court Judges of Cook County, one Commissioner being appointed each year to serve for a term of five years. Thus, it is practically a continuing body and is able to formulate a definite policy with a practical certainty of its being continued from year to year. The jurisdiction of the South Park Commissioners extends over practically all the corporate territory south of the Chicago River, 92.65 sq. miles, which includes the Loop, or central business district, and the south and southwest residence and industrial districts.

The Recreation Park has had its fullest development in the South Park System of Chicago, in fact, the Recreation Park of Chicago is often referred to as the model for the world. The City of Chicago cannot claim the honor of having originated the public playground or the social center, but it remained for that city to combine these two ideas with others to form a new and unique municipal feature in the Recreation Park.

Playgrounds are often rather barren enclosures with certain stereotyped play apparatus, perhaps a sand court and, sometimes, a wading pool. Social centers generally convey the idea of a somewhat plain, often unattractive building, administered with the uplift idea in the foreground. The Chicago Recreation Park is first of all a small park with trees, lawns, shrubbery, and many of the features of the formal park—balustrades, cement vases, sunken

areas, etc. Each park has a substantial building, generally of concrete with tile roof, of rather broad and rambling design, as much unlike a conventional schoolhouse or institutional building as possible. Each building contains an assembly hall, clubrooms, library and reading room, gymnasiums for men and women, with showers, locker rooms, and the usual conveniences of a public building. In some of the larger and more recent buildings there is a lounging room for men, and bowling alleys in the basement; in fact, the arrangements are designed to be as much like a club as possible.

The Public Library maintains a branch in most of the buildings with a trained librarian in charge. Outside the building, but in close connection therewith, is an open-air swimming pool of concrete which, in the later pools, is lined with terra cotta. There are also outdoor gymnasiums for men and women (or boys and girls), with fully equipped playgrounds for children, and as large a ball field as space will permit. All these features are situated in a park setting, often with pools, lagoons, and bridges.

A director is in charge of each building, with athletic directors of both sexes in charge of the social and athletic activities. The buildings are open evenings and Sundays. The assembly halls are, by assignment, used for dances, lectures, educational pictures—anything except for religious and political discussion.

Regular gymnasium classes are conducted, all clean sports are encouraged and taught, all kinds of contests are arranged, leagues formed, and trophies given. Events, such as kite tournaments, lantern parades, artificial flower shows, etc., are continually being devised and given a definite place in the season's program. By a cumulative system of marking in all contested events, the crippled boy who can make and fly the most successful kite, or the little girl with a knack of dressing her doll above the mediocre, is taught to feel as much pride and personal responsibility in the standing of the park organization as the athlete who wins the wrestling event. Clean sportsmanship, with the determination to win, is always the slogan. There is even more in wise administration of the Recreation Park than in skillful designing.

The South Park System of Chicago may be said to be entering on a third stage of park building. The first stage was the establishment of the original system of large parks and connecting boulevards; the second was the construction of recreation parks; and the third is the reclamation of the Lake Front of the southern half of the city and the building of a great parkway, or chain of parks, along this shore line, extending from the mouth of the Chicago River southward to the Indiana State line, a distance of nearly 14 miles, from which should be deducted 2.5 miles where the water-front is occupied by the steel mills.

The part of this work now under way is the 7-mile stretch from Randolph Street, the northern limit of Grant Park, to 57th Street, joining there the shore line of Jackson Park; also 1 mile of lake front from 95th to 102d Streets, south of the steel-mill district.

The project has often been alluded to as the "Outer Boulevard" or "Lake Front Drive." These terms are not correct, as the territory to be reclaimed from

the waters of Lake Michigan is more than 3 000 ft. wide in some places. There will be 1 600 acres included in the project between Randolph and 57th Streets, of which 1 200 acres will be land and 400 acres lagoon. Between 95th and 102d Streets, there will be 108 acres of new made land. When these two sections are complete, the area of the South Park System will be increased 60 per cent.

From 57th to 79th Street is a third section of the lake shore, part of which is Jackson Park frontage, which is not included in the plans for the immediate future, but is reserved for later consideration.

For many years the tracks of the Illinois Central Railroad occupied the immediate lake shore from Randolph Street to about 50th Street and, by reason of the possession of riparian rights, no other use of that part of the shore was possible.

In 1895, after a long period of litigation and negotiation, the limits of the rights of the Railroad Company between Randolph and 12th Streets were fixed by agreement, and the right of the City of Chicago was established to fill an area east of the railroad tracks to a harbor line defined by the Secretary of War. In 1896, the control of this area, then named Grant Park, was vested in the South Park Commissioners.

Filling progressed for several years, partly with city wastes and partly with material dredged from the Chicago River and Harbor. In this manner, in a period of about 12 years, 164 acres of land were made for park purposes, filled to elevations 7 to 28 ft. above water level, where the water was formerly 10 to 15 ft. deep.

For various reasons, like many other public improvements, the finishing of Grant Park has been delayed through a period of years, although plans have been made and changed from time to time as new conditions arose. In spite of its somewhat barren condition, many uses have been made of it, from year to year, for military spectacles, camping ground during the World War, aviation field, etc. It is now well under way to completion as a somewhat elaborate formal park suitable as a front-door yard of a great city.

In 1912, another agreement was made between the South Park Commissioners and the Illinois Central Railroad Company, whereby, in exchange for certain definite increase in width of right of way, the Railroad Company conveyed to the Park Commissioners all its somewhat intangible riparian rights from 12th Street to 50th Street. At the same time, the Railroad Company desired a permit from the City authorities to erect a new passenger terminal at 12th Street, and the City authorities desired the electrification of the railroad. As all these matters were closely related, the Park Commissioners were unable to realize at once their plans of building a new shore line to Lake Michigan. Other negotiations followed, political strings had to be pulled, and the whole matter was delayed.

Finally, in 1919, all parties reached an agreement, and an ordinance was passed in the City Council and accepted by the South Park Commissioners and the Illinois Central Railroad Company, providing for the erection of a new passenger station, electrification of all Illinois Central Lines within the city by a certain time, and confirming the agreement between the Railroad Company

and the Park Commissioners. The assent of the Federal Government was given to the combined project by a permit issued to the South Park Commissioners to fill in the shallow waters of Lake Michigan along the shore from 12th Street to 57th Street, a distance of 6 miles, and to a width of from 1 500 to 3 000 ft. Thus, the people of the south half of Chicago came again into possession of their lake shore which had been given away 75 years earlier to secure the entry of a railroad. The railroad would probably have entered in any case, and the City Council considered it was driving a sharp bargain to allow the new railroad to build along the shifting line of the lake shore to serve as a kind of barrier for the lowlands behind.

It will cost $50 000 000 or $60 000 000 to rebuild the shore line as planned, only $50 apiece for every present inhabitant of the South Side, and the Illinois Central Railroad Company will spend another $50 000 000 or $60 000 000 to complete its part of the closely related improvements. The general plan of the South Shore Park improvement is to extend the present shore line 400 to 600 ft. in width, and leave there a waterway, 400 to 600 ft. in width, the entire length of the improvement, and build another strip of land on the outer side of the lagoon.

All the shore lines of the newly made land will be formed by bulkheads to retain the fill. As far as planned and constructed to date, the bulkheads are of the so-called "pile pier" type, consisting of two rows of closely driven piling with wales and tie-rods, varied in detail according to conditions, filled between with stone. Details differ in various parts of the work as the finish of the shore line varies. At some places retaining walls will be built on the bulkheads, whereas, at others, low concrete docks will be built for boat landings, or beaches will be paved to the water line, or the banks rip-rapped, stony promontories, etc. Where walls, concrete docks, or pavements are supported on the bulkheads, a line of Wakefield sheeting is incorporated in the structure to prevent leakage of the earth or sand filling through the bulkhead.

Untreated piles and timber are used, as the plans require the cutting of all woodwork to the water level, when finished. To retain the fill temporarily, the piling and stone-fill are left at an elevation of 5 ft. above low water. Large blocks of rough stone, not less than 1 cu. yd. in size, are used to top off the stone-fill within the bulkhead. This stone will be removed and re-adjusted to the final grade or used elsewhere as the shore line receives its final finish. The water along the line of the outer bulkhead, permanently exposed to the lake, is 16 to 20 ft. deep. At exposed points and where the bottom of the lake is soft, banks of heavy rip-rap are used to reinforce the piling.

The piles are from 35 to 50 ft. in length, according to the depth of the water and other conditions. Where the depth of water is 16 ft. or more, the bed of the lake is generally clay which becomes gradually stiffer and harder with depth, until hardpan or boulders are found overlying solid rock at a depth of from 70 to 90 ft. Sand lies along the undisturbed shore lines above a 16-ft. depth of water. The conditions differ according to location, very soft clay streaks being found in some places, whereas, in others, particularly between 39th and 50th Streets, reefs of rock extend too near the bed of the lake. It may



FIG. 1.—AIR VIEW OF GRANT PARK, FIELD MUSEUM, AND STADIUM, CHICAGO, ILL.

be necessary to change the type of bulkhead at such places, possibly to cribs or stone mounds. As no construction has been undertaken where rock lies at such shallow depths, and as the investigation of the lake bottom is not complete, no other design of bulkhead has yet been adopted. Many suggested designs for bulkheads involving the use of concrete piles, steel sheeting, concrete caissons, etc., have been examined, but as yet no plan has been developed, which can compete with the wooden-pile, stone-filled structure in cost or flexibility of design for the purpose. Wood beneath water will resist the action of the elements indefinitely, and there are no organisms in the waters of the Great Lakes, that will attack wood.

About 20 000 lin. ft. of bulkhead has been constructed in three separated sections, the plans being to conduct the work from as many different points. During 1922, 1 500 000 cu. yd. of filling were deposited by contract, and each year, at the present rate, about 1 000 000 cu. yd. are received from city wastes, miscellaneous excavations, etc. Filling from the latter sources is obtained at no cost to the Park Commissioners, except that of maintaining dumps and dressing to grade. Most of the filling required, however, will be obtained by dredging operations from the bed of the lake. After the filling has been completed, many details remain to be done to produce a finished park territory. A number of farms will be stripped of top soil to cover the new made land to support trees, shrubbery, and lawns.

Besides the usual functions of a city park, this great Parkway under consideration will serve the people in many unusual ways. The lagoon or enclosed waterway is 6 miles long from Jackson Park to Grant Park, 400 ft. wide at the narrowest places, spanned by numerous bridges, and will afford unrivaled opportunities for boating, canoeing, and other aquatic sport sites will be provided for boat clubs and public boat-houses will be erected. The carrying of passengers to and from the city by barges will be encouraged. What can be more invigorating than to travel to and from business and shopping by such means of transportation, unless it is by motor along one of the two main driveways bordering the open lake, where there are no crossings or business traffic? The time by motor car from the Hyde Park residential district to the business center will be cut almost in half when these drives are completed.

There will be at least four large bathing beaches established along this stretch of new lake shore, the first of which, at the southern end in Jackson Park, containing in the beach house 6 000 lockers, is already in use. To insure purity of water, it was determined that all bathing beaches must be entirely open to the lake, without any obstructions or barriers whatever in front.

To build a beach to meet these conditions in 20 ft. of water was a new problem in engineering; in fact, bathing beaches are generally not built. The plan adopted, after much study of natural and accidental conditions along the lake shore, is to construct the main bulkhead along the outer shore line so that, after the general fill is completed, it may be cut down to 2 or 3 ft. below water for a length of 1 000 or 1 200 ft., to serve as a submerged reef and barrier to retain the sand slope of the artificial beach.

The waves will trip over the submerged barrier, perhaps dig to a depth of 5 or 6 ft. behind it, and expend their remaining energy as on all natural beach

slopes. Below water level, the average slope of the sand on Lake Michigan beaches is about 1 on 40. On this basis, the water-line at the proposed beaches should be from 200 to 250 ft. back of the bulkhead line.

None of the proposed beaches has been built according to this plan, but a contract including the first one will be awarded early in 1923. Although there may be unexpected results, careful studies have been made, and it is believed the plan is sufficiently flexible to meet all contingencies.

The time required to complete the entire project as described can scarcely be stated, as so much depends on the financial arrangements. Legislative authority has been obtained for the issuance of bonds to cover the entire estimated cost, and they will be issued from time to time, according to the needs of the work, each issue being submitted to a vote of the Park District. The first issue was for $8 000 000, which will be nearly expended by the end of 1923. No estimate of the time required to complete the project has been made less than fifteen years. Two years have already been actively spent.

The first objective to be attained is to provide a new route from the central south side to the down-town district by a new drive on the new made land from 23d Street to Grant Park. This involves the construction of a viaduct 634 ft. long between abutments (about 1 600 ft. long, including approaches) and 120 ft. wide, across the tracks of the Illinois Central Railroad at 23d Street; also, the widening of South Park Avenue to 198 ft. from 23d Street to a junction with Grand Boulevard at 35th Street. The cost of the viaduct and approaches is estimated roughly at $1 250 000 and that of widening South Park Avenue, $1 000 000, exclusive of the cost of property to be acquired.

It is expected that the viaduct and widened South Park will be ready for use in the latter part of 1924. By that time, the filling and driveway on the new fill will be complete from 23d Street to Grant Park.

Other prominent features connected with the Lake Shore project are the Field Museum at 12th Street, built with funds provided by the will of the late Marshall Field, and the Chicago Stadium now being erected by the South Park Commissioners, just south of the Field Museum. Both these structures are on filled ground. The Field Museum has been open to the public for more than a year. It is built of white marble, and is one of the largest structures of the kind in the world. Fig. 1 is an air view of Grant Park and its surroundings.

The Stadium is of reinforced concrete with artificial stone exterior walls. Its arena is 1 000 ft. long and 300 ft. wide, and the seating capacity when finished will be 60 000.

An interesting feature of the Lake Front project is that this is the first time in the history of park building that plans have been especially made for the entry of street-car lines into the park. Provision is made to carry all cross-town lines over the Illinois Central Railroad and the lagoon on viaducts.

Car lines have been carried into or through other parks by force of circumstances, but not as a part of deliberately made plans.

ZONING—ITS PROGRESS AND APPLICATION

By Morris Knowles,* M. Am. Soc. C. E.

Synopsis

The object of this paper is to show the engineer the scope and effect of this relatively new method of control of the development of private property. Zoning is really an important part of city planning. Just as city planning is not a single profession, neither is zoning, but rather the result, where best worked out, of the co-operative effort of a number of professions under wise leadership.

Like city planning, however, it is of the greatest importance and interest, particularly to the municipal engineer. He is concerned with the backbone of city planning, that is, the street layout, thoroughfares, block arrangements, utilities of all kinds, and planning of houses, stores, and factories. Thus, he is especially to be active when it is planned to control, regulate, and even fix some of this development by man-made laws.

This paper outlines the rapid growth of zoning, since its introduction in New York City in 1916. Next, the dangers that may result from its evident popularity are mentioned. A definition of zoning and statements of what can and can not be accomplished by it are next presented. The need of the trained legal mind is apparent throughout all steps, both in the preliminary legislation and in the enactment and enforcement of the ordinances.

The work of the Hoover Zoning Committee, in collating present legislation, is stated briefly, together with the outline of the Standard Enabling Act, recommended for the consideration of all Legislatures when contemplating such laws. The need of information on present conditions, in the attempt to plan wisely for the future development of the municipality, leads to a realization that city planning is the main purpose, of which zoning is only a part; zoning is the attempt to render the whole a stable creation.

The types of regulations and subjects covered, although of varying detail, can be classified generally under three main headings: Use of property and structures; heights to which buildings may be built; and areas of lots which may be occupied. Such kinds of drastic legislation need safety valves. One is provided by a Board of Adjustment or Appeal and another by the means for making necessary amendments, with such safeguards as experience has shown to be wise.

The summary next follows, with the conclusion that the proper aim of zoning is usefulness, and that education and publicity are needed, in order that the demand may be general and result from popular appeal. Finally, the real test is the approval of all the people that it is worth while.

Progress

New York City Example.—It is peculiarly appropriate that this subject should be discussed in the great city where it originated. The first complete

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zoning plan in the United States—embodying all the general features as they are now understood—was adopted in New York City in 1916. This was made possible by legislative amendment of the New York Charter in 1914. Additional amendments were made in 1916 and 1917, and zoning was made of general application in the State by later Acts. Prior to this time, under the Acts of various States, only the less complete kinds of zoning were possible.

The New York story of how zoning has developed and how the interest has grown is perhaps a most striking example of the influence of the popular will, urgently and effectively presented. Before the passing of the zoning law, there was great agitation because of lack of regulation of congestion and the encroachment of industry in neighborhoods thought to be desirable for other uses. There was a realization that something must be done. Such interest centered around Fifth Avenue—the brightest business street in New York City—as it was on the way to ruin because of the invasion of workshops for garment and millinery workers, and similar industries. Many such establishments were drawn to this Avenue because of the attractiveness of its buildings, nearness to the department stores, for which the goods were being made, and the advantage of a Fifth Avenue address. The result was that crowds of garment workers thronged the Avenue, morning, noon, and evening, crowding out the shoppers and the hotel population.

This resulted in the formation of a protective association, comprised of the owners of the leading department stores, businesses, and hotels. Long before there was a zoning ordinance in New York City, there was an edict from the Executive Group of this Association, to the effect that preference in purchases would be made, by the department stores and members of the organization, of goods manufactured outside a given area or zone. This zone extended from Third to Seventh Avenues, and from 33d to 59th Streets. Manufacturers who were within this zone were given eleven months to withdraw, and at the end of that time it was evident that business would not flow to those who stayed. Two full-page notices appeared in the New York daily papers in March and April, 1916. The first was the edict, as follows:

(From *The Evening Post,* New York, March 6, 1916.)

"SHALL WE SAVE NEW YORK?

"A Vital Question to Every One Who Has Pride In This Great City

"Shall we save New York from what? Shall we save it from unnatural and unnecessary crowding, from depopulated sections, from being a city unbeautiful, from high rents, from excessiye and illy distributed taxation? We can save it from all of these, so far at least as they are caused by one specified industrial evil—the erection of factories in the residential and famous retail section."

* * * * * *

The second, showing a significantly willing compliance therewith, was as follows:

(From *The New York Times*, Sunday, April 2, 1916.)

"SHALL WE SAVE NEW YORK?

"The co-operative movement to preserve the heart of the City, from 33rd to 59th Streets and from Third to Seventh Avenues, from destruction by factories and to rehabilitate the lower and deserted part of New York City, has met with an unprecedented response from citizens, business firms and other organizations, not only from every part of this city but from all over the United States. The public has put itself squarely behind this great civic movement. One of the most significant evidences of the general acceptance of the plan is illustrated by the unselfish and public-spirited attitude of the cloak and suit manufacturers."

(Then followed a petition by the Cloak, Suit and Skirt Manufacturers' Association, signed by the leading firms of the city, endorsing the movement and requesting all citizens to support it.)

The effect was even as marked as the result of formal legislation, and the prohibition thus started was soon made effective in ordinance form, after the report of the New York "Heights of Buildings Commission" and another one on the "Congestion of Population". There is now a new center of clothing and similar industries on the West Side, in convenient, commodious buildings, suitably planned and adapted for these purposes and without detriment to other city facilities. This is a wonderful example of bringing about, by public sentiment first, and later by the same principles being enacted into law, the regulation and control of the use and development of property in the greatest metropolis of the world.

Growth.—Since the enactment of the New York law, zoning legislation has been passed in twenty-seven States and the District of Columbia. In twelve of these States the Enabling Act is comprehensive and permits zoning of all forms in all municipalities. One hundred and fourteen cities, ranging in population from nearly 6 000 000 to a few thousands, have, to a greater or less extent, established zoning plans. Eighty-one cities have comprehensive ordinances, of which thirty-one relate to use and two to height. In the States where zoning is possible, many more cities are preparing to take advantage of the opportunity thus offered; and in the States where zoning is not possible, preparations are being made for the introduction of the necessary legislation. It would seem, therefore, that experience thus far is demonstrating the value of zoning; that the idea is steadily growing in popularity; is more or less general throughout the country; and the indications are that it will become more so.

A word of caution, however, may be advisable lest its popularity be the means of obstructing its true progress, through well-meaning but too hasty action. Zoning of real value cannot be done hastily and without adequate study and investigation; consideration must be given to local physical conditions, opinions, and usage, and to all the surrounding circumstances. These factors are important in determining what will be useful—what kind and degree of protection is needed and how it shall be applied. What is good for

one city may not be suitable for another, even if the general principles of zoning are the same. Advantage, of course, should be taken of the experience of other cities and the opportunity to review ordinances, but such study must recognize the reasons that have prompted the different methods in different places. A plan for one city cannot be adopted intact for another, and any attempt to do so may prove to be disastrous and a serious set-back to true progress.

Definition.—Technically, zoning is "the creation by law of districts in which regulations, differing in different districts, prohibit injurious or unsuitable structures and uses of structures and land."

Practical Purpose.—Practically, the aim of a zoning plan is to establish a basis for constructive city growth. All cities have the same general features. They are made up of streets, parks, and other public places, and of land devoted to houses, stores, factories, and similar private uses. Unity in construction, however, is not possible unless there is some measure of control over all land, whether publicly or privately owned; and the only practical method of acquiring such control is through a zoning law.

Such control will promote greater economy, convenience, safety, health, and comfort in industrial, business, and housing conditions. It will guarantee a definite and safe place for industrial investment, will protect home neighborhoods, stimulate home ownership, and assure more contented labor conditions. It will stabilize property values, afford greater security for mortgage loans, and a surer basis for assessment.

What Zoning Does not Do

A.—Zoning does not endanger or retard the progress and general development of any community, large or small.

B.—It does not discontinue or annul private restrictions and deeds.

C.—It is not discriminatory, and does not infringe on constitutional guaranties.

D.—It does not change existing conditions. It is not retro-active and does not prevent the continuance, under present conditions, of existing non-conforming uses or structures.

E.—It does not establish permanent and inflexible provisions that cannot be changed later if necessitated by normal development and growth.

What Zoning Will Accomplish

A.—Zoning encourages prosperous and well-organized community growth.

B.—It makes possible a practical program for future street development and for all public utilities, by determining in advance the future use and requirements of all districts.

C.—It prevents too sudden changes or conversions in the character of districts.

D.—It prevents the intrusion of inappropriate buildings or uses in districts where such buildings and use would injure established improvements.

E.—It stabilizes and protects values and investments by determining in advance the uses to which property may be put.

F.—It simplifies the problem of traffic by regulating the height and bulk of buildings, and the consequent street congestion.

G.—It prevents undue congestion of population by regulations to prevent overcrowding of land.

H.—It insures better conditions for health and sanitation in offices and factories, and minimum requirements for light and air.

Interest of the Engineer

Many interests are concerned in the development of a modern city—the politician, the lawyer, the engineer, the architect, the landscape designer, the business man, the representative of industry and labor, the economist, the student of social problems, the builder, the real estate broker and owner, the banker, the insurance man, and others. Each has his place and his particular point of view in shaping public policy.

It is the function of the engineer, however, to plan the physical features of development: The transit lines, the transportation facilities, the public utilities, the housing and sanitary features. He plans the industrial and business features as well as the physical future of the town. To do this efficiently there must be available a predetermined general scheme or arrangement—one that will remain reasonably permanent. The importance of this is self-evident.

A zoning plan, then, is the basis on which all other plans should be conceived; and it may be said that the zoning plan is primarily an engineering problem. No wonder, then, that the engineer is interested. This is true, however, only in so far as its essential features are concerned; the plan will be of no avail unless established on a firm legal basis; it will be acceptable only if the point of view of all other public and private interests has been considered in its preparation.

Assurance that the plan will stand the test of Court review can be had only through co-operation with attorneys capable of interpreting the meaning of provisions and legal decisions made thereon. Fine distinctions, that escape all but those specially adapted to perceive such refinements, are often made. These distinctions are influenced many times by special circumstances surrounding a particular case, and may have a different interpretation under different conditions.

The zoning plan can be conceived with sufficient breadth of view only through counsel with and co-operation from all interests affected.

Legal Aspects

Zoning is possible under the exercise of the "police power" of the State. In a broad sense, the police power of the sovereign authority embraces whatever may be best for the general public welfare of the Commonwealth. Another name—an easily understood equivalent—is "community power".

Police Power.—No attempt has been made by the Courts to define the limits of the police power—it cannot be done adequately—necessarily, what is for the public advantage may vary according to location, local customs and

usage, physical conditions, etc. The influence, therefore, of public opinion on what is for the general welfare is recognized by the Courts. An examination of the various Court decisions will show this to be true, and also that the police power has been extended so as to sanction its application beyond former limits. In the case of the Noble State Bank *v.* Haskell, 219 U. S., 104 (1911), the following statement by the Supreme Court of the United States is made:

"It may be said in a general way that the police power extends to all the great public needs. It may be put forth in aid of what is sanctioned by usage, or held by the prevailing morality or strong and preponderant opinion to be greatly and immediately necessary to the public welfare."

Again, in the case of Eubank *v.* Richmond, 110 Va., 749 (1910), 226 U. S., 137 (1912), the following statement is made:

"That power [the police power] we have defined, as far as it is capable of being defined, by general words, a number of times. It is not susceptible of circumstantial precision. It extends, we have said, not only to regulations which promote the public health, morals and safety, but to those which promote the public convenience or the general prosperity."

In the application of zoning regulations, more restrictive measures in single-family residence districts than in general residence districts (containing apartment houses and similar structures) may not be solely justified on the grounds that the former are more certain to insure public health, safety, welfare, etc., than the latter. It is evident that the measures designed and suitable for the single-family residence districts, if applied to the general residence districts, would interfere seriously with existing development and would set up such a discrimination, between property already improved and that to be improved, that their wisdom or expediency would be doubtful. On the other hand, more lax regulations, suitable for the general residence districts, if applied to the single-family residence district would be useless and would permit of great depreciation of property values, that is, "general prosperity".

The application of restrictions, to be justified, must be reasonable, and districts of each type or character, as determined by their development, are treated according to their particular needs. In more general terms, regulations, to be equitable and to be acceptable or to come within the scope of the police power, must affect all similar properties or similar districts throughout the city in a similar manner.

Enabling Act.—As a general rule, cities are not able to establish zoning regulations without some special grant of the police power from the State. Such acts usually prescribe the character and extent of the regulations which may be imposed by the municipality, the limitations of power, the imposition of penalties, and provisions for appeal and amendment.

Some cities have what is called a "Home Rule Charter"; this is particularly true in some of the Western States. Such charters sometimes give the city the authority to zone, by virtue of the grant of charter powers, and special legislation may not be needed for this particular purpose. However, in such circumstances, it is wise to make sure that the city possesses the police power to regulate the height, bulk, area, and use of buildings and property, and the

right to impose different regulations in different districts of a different character.

The Standard Enabling Act, as recommended by the Hoover Committee of the United States Department of Commerce, provides a complete example of the legislation necessary to delegate police power to the municipality, in order to regulate the uses and development of property and provide all the machinery for doing so.

Section 1 recites the grant of power, with definite limitations and expression of purpose. Section 2 permits the division of the municipality into districts, providing that regulations throughout each district shall be uniform, but the regulations in one district may differ from those in others. Section 3 expresses the general purpose of such legislation, and makes clear the establishment of the "atmosphere" under which zoning is to be done.

Section 4 provides for the manner of the creation of the districts and, together with Section 5, the method of making changes and amendments, and the provision that certain percentages of interested persons may necessitate a vote of three-quarters of the municipal legislative body in order to amend. Section 6 provides for the creation of a special body to be known as a Zoning Commission (or its equivalent), working as a City Planning Commission, for the purpose of making the original study, holding hearings, and presenting a report to the municipal legislative body on the subject of a zoning ordinance.

Section 7 provides for a Board of Adjustment or Appeal, its membership, rules of procedure, machinery of appeal, and further appeal to the Courts. It is provided that the Board of Appeals shall review any act of the administrative officer on alleged error. There are also certain listed exceptions from the terms of the ordinance which such Board may permit.

The Board also has the power to vary the terms of the ordinance where, owing to special conditions, the literal enforcement will result in unnecessary hardship, but, at the same time, the spirit of the ordinance shall be observed and substantial justice done.

Section 8 provides the remedies and penalties for violation of the ordinance. Section 9 states that where there is conflict with other laws, the higher standards and restrictions shall govern.

Ordinance.—The regulations of a city are established and applied through the usual ordinance or legislative machinery, drawn in accordance with the provisions of the State Enabling Act. Such an ordinance usually consists of one or more maps dividing the city into different kinds of districts and statements of the regulations to be applied in each district regarding use, height, and area.

Legal Advice Needed.—The success of a zoning plan depends on the adequacy of the Enabling Act, and the provisions of the ordinance must be such as to come within the limits of the Act and within the scope of the police power. There is need, therefore, of the lawyer who is familiar with and can rightfully interpret the decisions of the Court and the probable attitude in specific instances. Some features of zoning are not yet approved by the Courts so widely as to be generally recognized as coming within the scope of the

police power. The greatest stress, therefore, should be laid on the importance of having sound, experienced legal advice in the drafting of the ordinance.

Preparation of the Zoning Plan

Planning Commission.—There should be a commission to prepare the zoning plan and ordinance for recommendation to the City Council. This may be the City Planning Commission, if there is one, or a special Zoning Commission appointed for this purpose. The majority of the members of such a commission should be, in most cases, citizens serving without pay. They should not be hampered by political considerations. The City Administration, however, should be represented, and this usually is accomplished by certain of the officials acting in an *ex officio* capacity. Actions of immediate expediency are thus to be given due consideration, as well as the effect on the far-reaching future.

The creation of such commission, the membership, method of appointment, and powers, are usually provided through the statute or charter provisions. The commission should have more than advisory powers; usually, it is provided that, before any other city authority acts finally on an amendment to any clause of the zoning ordinance, it must be referred to the commission for consideration, and a certain time allowed for a report.

Working Staff.—As the members of such a commission cannot attend personally to the collection of data, the preparation of maps, and the working out of details in the development of the zoning plans, they should be provided with an adequate staff of trained men under the direction of a competent executive.

In addition, they should have the benefit of expert experienced advice. Zoning requires the services of more than a single profession. It requires knowledge of all those things which go to make up municipal development—the proper planning of streets, transportation facilities, public utilities, housing, etc. The services of the all-around civist are needed; but here a word of caution may not be amiss, especially if the commission or staff is inclined to be lazy or lacks diligence. There is real danger, if the expert, however well informed, attempts to foist on a community, without the detailed study of local needs and application of principles derived therefrom, a plan that has worked well elsewhere. The ordinance must also appeal to the local sentiment, and it cannot do so without being a result of local education and development.

Reliable Data Required.—Complete and accurate data are needed, and an intensive study of local conditions and problems is required. Such data will refer to the use and height of buildings, the vacant spaces about them, the character of industries and establishments, the population and densities thereof, the values of land, the location of business enterprises, the tendencies of development, etc. The data should be presented, preferably, in graphical form or on maps, so that they may be comprehended and analyzed readily.

General Procedure.—The commission, after completing the preliminary plans and tentative form of the zoning ordinance, should hold meetings and public hearings prior to submitting the completed work to the Council. The Council, after receiving the ordinance, may hold other meetings, and, if it is

deemed necessary, may refer the ordinance back to the commission for amendment, or may enact it into law.

It is not possible to emphasize unduly the need of a thorough public discussion of the zoning plan. To be acceptable, the ordinance must meet the approval of substantially the entire community, and must demonstrate its reasonableness to property owners. If the general public is to be able to understand and consider the merits of the plan seriously, it must know why certain things have been done and the reasons for certain regulations. This can be accomplished best through frequent meetings during the work, particularly before small groups where peculiar local problems may be discussed and where the protection offered to residence property may be explained. Similar meetings should be held for discussion with the owners of business property, with manufacturing interests, real estate associations, and the various civic bodies of the community. The influence of the press will be of great assistance, and frequent articles should be published, explaining the purpose and the effect of the various regulations.

Zoning Regulations

General.—A complete zoning plan utilizes three general methods of regulation: The use of property and structures; the height to which buildings may be built; and the areas that must be left vacant.

Use Regulations.—The purpose of the regulations of the first class is to segregate the various uses of public and private property and restrict such uses to suitable locations. Some land will be found to be especially fitted for certain purposes, industries will naturally be placed near railroads, or near the waterfront; residence districts of the better class will develop remote from industry, and not too near the business section; and workingmen's houses will be built near their work, or near adequate transit lines. The types of districts in a city are usually well-defined, and are more or less fixed by existing development, topography, and cleavages, like watercourses or hills.

Zoning should anticipate the continuance of established uses of property, when these are not in conflict with wise development, even if the ideal arrangement might possibly be better. The city cannot be rebuilt overnight, and it is possible only to approach the ideal. Advantage must be taken of the mistakes of the past, but the harm that may already have been done by indiscriminate building is of small account if the future of the city can be protected.

Restrictions as to use are the most important provisions of the ordinance and those most readily understood. In general, there should be regulations for heavy industry, light industry, business or commercial districts, and residential purposes. Some communities will deem it necessary to divide industry into three groups, or to have a secondary commercial district for light manufacturing, and some will have three classes of residence districts. In the speaker's opinion, such minutiæ are undesirable, and are impracticable of exact definition, at least at the inception of the ordinance. Such detail, if utilized at all, should come after the workings of the provisions of the ordinance have been tested.

In some cities, there are no limitations on the use of structures or property in the heavy industrial districts, but usually certain manufacturing processes which create nuisance or which are dangerous, are prohibited. Dwellings in such districts should not be permitted, except under definite limitations, such as provisions for the use of watchmen employed on the premises of an industrial establishment and their families.

The light industrial districts are intended for factories which are not objectionable because of odor, dust, smoke, gas, vibration, or noise. In addition, business establishments of any kind are permitted, and residences.

The business or commercial districts are intended for warehouses, wholesale and retail stores, office buildings, hotels, and similar structures. No manufacturing is permitted, other than that required for products to be sold on the premises to the consumer. Other establishments which would be objectionable, such as blacksmith shops, carpet-cleaning works, contractors' plants, public garages (except with special permits), etc., may also be prohibited.

Residence districts should allow single and multiple-family dwellings, churches, educational and charitable institutions, greenhouses, gardens, and the like, including their usual accessories. Public utilities, by special permit, when found to be necessary (such as telephone central exchange buildings, electric sub-stations without rotating machinery, and gas-regulating stations), may be permitted in the least restricted districts and in some cases excluded. The accessory uses are so varied and so numerous that it is difficult to recount them. It is safer to refer to the classification as "those accessory uses which are customarily incident to the uses permitted in the district, and not involving the conduct of a business."

The conditions under which small garages and stables and community garages are to be permitted, should be specified carefully. It is possible, also, to curtail the erection of bill boards, signs, and spite fences by zoning regulations. Although bill boards may be proper in certain places, the Courts have recognized that they should not be placed indiscriminately among houses, schools, and churches. Spite fences may be controlled by limiting the height of fences.

Many zoning ordinances have provided two or more classifications for residence districts, differing only as to the restrictive regulations. Primarily, the purpose is to provide for single-family residence districts. This may be requested by large numbers of people, and may be desirable, but, legally, it may be hazardous unless the power is expressly stated in the Enabling Act. It may be difficult, in many cases, to justify such procedure on the basis of health and safety. However, the definite limitations of certain districts to the single-family house is becoming more common in zoning ordinances, and the generally favorable attitude of the Courts has been reflected in recent decisions. It has been decided that the police power can be used for the purpose of establishing private residence districts, and it is only a step further to the classification of such districts.

Exceptions to the regulations pertaining to use, in district classifications, may be provided in the ordinance. Public and community garages, for

instance, are sometimes desirable in residence districts, and can be located so that they will not be detrimental to adjoining property. Exceptions of this kind are usually made contingent on the desires of the majority of the owners of property in the immediate vicinity. Provisions may be made also concerning uses which do not conform to the standards of the districts in which they are situated. Such uses may continue until they are discontinued voluntarily, or by fire or other destruction, or are altered structurally, or to a certain large percentage of value.

Height Classifications.—Regulations of the height of buildings, particularly in the business sections of a city, are likely to meet with the most active opposition. There is reason for this opposition, and careful consideration and study are required if an equitable balance is to be established between what is theoretically desirable and what is practicable of accomplishment. Office buildings, hotels, and similar structures must be built to a certain height if they are to pay adequate returns on the investment. Such buildings, however, may readily be built too high for economy.

It is well known that many of the lower stories of the high buildings in the canyon-like streets of New York are useless for practically all purposes except storage. In areas where high buildings are crowded together, most of the rooms on the lower floors are inadequately lighted and insufficiently ventilated, traffic congestion is inevitable, public utilities are periodically overtaxed, and both private and public financial loss is the result. There is a definite and real need, which the engineer is qualified to fulfill, to obtain and present carefully digested facts on the existing conditions of high and moderately high buildings as to light, air, comfort, waste, elevator and corridor space, street congestion, burden on public utility facilities, etc.

The report of the "Height of Buildings Commission",* discloses surprising facts regarding the average height of buildings in lower New York City:

"The high building problem is at present confined chiefly to a comparatively small portion of the lower half of the Island of Manhattan. The average building height in the Borough of Manhattan is 4.8 stories. Ninety per cent. of the buildings do not exceed a height of six stories. The buildings over ten stories in height constitute only a little over one per cent. of the total. Out of a total 92 749 buildings, there are but 1 048 buildings over ten stories in height; ninety buildings over seventeen stories; fifty-one buildings over twenty stories; and only nine buildings over thirty stories."

The average height of buildings in the down-town business section of Pittsburgh, Pa., where the higher office buildings, the larger department stores, and the prominent hotels are situated, is only a little more than three stories. In a paper† by Mr. H. J. Burton, of Minneapolis, Minn., it is stated that:

"The corner of Wall Street and Broadway was considered the most valuable land in America. It was about 40 ft. on Wall by about 40 ft. on Broadway, and originally had thereon about a six-story building, which earned a moderate profit, after paying the taxes on the assessed valuation.

"The owner of the L-shaped land, in the rear from Wall Street round to Broadway, erected a tall building thereon, using the light and air over the low building on the corner. He had $1 000 000 money and borrowed $1 600 000

* New York, December 23, 1913.

† *Proceedings*, National Assoc. of Building Owners and Managers, 1918.

more to pay for his new building, and earned a moderate net profit from his rentals, after paying his taxes and interest on mortgage. So the man on the corner concluded to tear down his low corner building and erect a tall building, to help pay his increasing taxation. This building took away part of the light and air from the rear building, which presently lost most of its tenants to other new buildings. The $1 600 000 mortgage was subsequently foreclosed, the mortgagor losing his title to the land and his additional $1 000 000 investment in the building.

"The man on the corner did not receive as much net income (after amortizing the cost of his new building) as he originally received on his little low building."

In this same paper, it is stated that:

"From 1913 to 1917, the assessed land values, exclusive of improvements, on Manhattan Island south of Fortieth Street, declined more than $186 000 000."

The statement* of a New York authority on the subject is quoted as follows:

"In my opinion after ten years' service, if there were no buildings between Forty-second Street and Broadway, of more than 8 stories, the land value would be more than it is to-day."

The height regulations are not of so great importance in other than the more congested business sections, where land values are excessive. The usual limit in the residence district is equivalent to a height of 2½ or 3 stories. The intermediate districts allow the buildings to be erected to heights of 4, 6, and 8 stories. All these limitations may be expressed in terms of multiples of widths of streets, number of stories, or in some combination of these, but it is rare that all are used in the same restriction. The maximum restriction varies from 125 to 150 or 175 ft., depending somewhat on the size of the city, with higher maxima with set-backs in New York City. Here, again, as in the use districts, exceptions may be made. They apply, in some cases, to institutions, hotels, public buildings, towers, spires, etc. Usually, also, additions to the height of buildings above the prescribed limits are allowed, if set-backs from the street lines (in some cases property lines, also) are observed. There is great need of data on the effect of height on the return from the investment in buildings, and the effect on the development of ample street areas and adequate public utility facilities. One is a private and the other a public effect, but reflected in a private way by taxation. This is indeed a fertile field for the engineer, for there is much opinion not founded on basic truth.

Area Regulations.—Area regulations may be applied in a number of ways, such as front-yard requirements; rear-yard depths; width of side-yards, inner and outer courts; percentage of lot occupied, and allowable density of population. Such regulations at best are complicated, and all methods are seldom used in one ordinance. Requirements designed to control density of population directly are the most interesting, and may be expressed as the number of families allowed per acre or the number of square feet to be provided for each family.

* *Proceedings*, National Assoc. of Building Owners and Managers, 1920.

Provision also should be made in the ordinance, if possible and legal, for continuing in effect the "set-backs" from the street, where already established by private restrictions or otherwise. This may be done by prescribing the depth of front yards, or the maintenance of a building line, as already established by a large percentage of the structures now built.

In connection with the area regulations, certain exceptions will be necessary, so as not to prevent the erection of a one-family house on a lot that may be too small to provide the required area. It is also necessary to differentiate between inside lots and corner lots, to provide special regulations for through lots, etc.

Administrative Agency.—Zoning ordinances, as a rule, are enforced by the same agency that has to do with building codes and other forms of building regulations. It is desirable to centralize the control of building regulations, so that permits for all features of construction may be obtained at one time by visiting one office. One of the serious difficulties and causes of annoyance in the building industry is the multitude of offices which must be visited to secure the many needed permits before construction can commence. Anything which can be done to simplify this and to lessen the expense will receive approbation.

Amendments and Changes.—The fundamental purpose of a zoning law is to prevent too rapid changes which result in the deterioration and modification of property values. Nevertheless, conditions will change, and different regulations will be required. Sometimes the people in a district believe the time has come for a change in use or classification. It may develop that, because of expansion of the community, or from some other reason, industry or business is best fitted for a certain residence district and that land will sell at a higher figure for such purposes than for any other. Then there will develop a popular demand for revision of the classification, and the same authority that has adopted the ordinance should have the power to amend the regulations.

Proposed changes should be studied first and reported on by the Planning Commission. A public hearing should be held to consider the requests for and the protests against amendments. This should be after the prescribed publication of notice and hearings and the report of the Commission to Council. If, then, the request for change is still opposed by a certain proportion of the property owners affected, more than a majority vote of Council should be required for approval of the amendment. This safeguard is wise, and is designed to prevent changes and amendments being easily made because of sporadic, whimsical, and changeable desires in particular cases of supposed and yet limited benefits to be gained by such amendments.

Board of Adjustment.—Because a zoning ordinance is so comprehensive, it is evident that hardship and serious discomfort might be caused by a strict and literal, but unreasonable, interpretation of its provisions, as they cover so many different things in a different way. It is desirable, therefore, to provide a Board of Adjustment or Appeal to which the decisions of the administrative agencies may be appealed and reviewed. Every decision of the Board of Appeals should be reviewable by the Courts on a writ of *certiorari*.

The Legislative Act granting to the city the right to zone should also make provisions for the creation of such a Board of Appeal, otherwise the City Council cannot endow such a board with powers to decide certain cases or to make variations in the provisions of the ordinance in order to carry out the spirit of the law and prevent unnecessary hardship. The province of the Board of Adjustment should be definitely and carefully limited to those things prescribed in the Act; otherwise, there may be gross abuse of the power thus delegated. That there is danger, however, should not deter one from providing such a board or safety appliance; for otherwise the ordinance may fail in its purpose by being declared unnecessarily harsh and confiscatory whenever it is brought before the Courts on a particular and exceptional case.

An example of reasonable variation is that the use of premises may be permitted for the manufacture of building material in a territory where it is evident that this is a convenient and economical way to prepare local materials for the erection of buildings within the district. There may be border line cases, also, regarding public service, enlargement of non-conforming use, continuance after fire or disaster, etc., which can best be handled by such Board of Adjustment.

There may be other types of exceptional cases which are difficult of interpretation by the administrative officer. Provision should be made for these and all others, where there are practical difficulties or unnecessary hardships in complying with the strict provisions of the ordinance. In such cases, the Board should vary and adjust in harmony with the general purposes and intent of the ordinance, so that health, safety, and general welfare may be secured and substantial justice done.

The following is quoted from the brief of Edward M. Bassett, Esq., as *amicus curiæ* in the Sheldon-Astor New York case:

"In the more complex field of regulation of all buildings and their uses, it would be dangerous if the State legislature did not provide some method of adjustment in exceptional applications for permits. The Board of Appeals is the safety valve in the zoning plan. Its function is analogous to that of a regulatory commission for public utilities, or the pure food administration under the federal statute. The legislature establishes a rule within which the administrative board or commission shall regulate. Rates shall be *reasonable*. Service shall be *adequate*. Appliances shall be *safe*."

In this connection, it is of interest to quote from the decision of the Court of Appeals of the State of New York in this same case:

"The courts below held that no zoning regulation adopted by the board of estimate and apportionment may in effect be repealed or set at naught by the board of appeals by action taken in the guise of a variance. Such conclusion is not only in conflict with the expressed intention of the zoning statutes but likewise the power delegated to the board of appeals directly by the legislature and by the board of estimate and apportionment pursuant to legislative authority."

* * * * * *

"The authority conferred by section 7 upon the board of appeals was to 'determine and vary' the use district regulations, by section 20 to 'vary' any

provision of the resolution in a specific case, where substantial justice will result therefrom."

* * * * * *

"If as the result of investigation the board of appeals shall 'determine', as it did in this specific case, that there existed unnecessary hardships in the way of carrying out the strict letter of the provisions of the zone resolution and that substantial justice would be promoted both to the property owners and the public interest, its power then was enlarged to 'vary', *i. e.*, to modify or alter in form or substance the application of the regulations of the board of estimate and apportionment in the specific case, which it did by permitting the building which the owner proposed to erect upon his lots embraced in the business district to be extended so as to cover the lots fronting on Madison Avenue."

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"Concluding as we do that the legislation under consideration authorized the board of estimate and apportionment to confer upon the board of appeals the authority exercised by the latter board in the instant case, and that the amendments to the regulations, a part of the zone resolution adopted by the board of estimate and apportionment, were in consonance with the legislative intention, the orders of the Special Term and Appellate Division should be reversed and the writ of *certiorari* dismissed, with costs to appellants in all courts."

Summary

Practical zoning must be established on a firm legal basis. A proper grant of the police power of the State to the city is pre-requisite. The preparation of the ordinance so as to be safely within the scope of the police power thus conferred is equally important. Therefore, the advice of a lawyer fully informed on the various decisions of Court cases is needed.

In the preparation of the plan and ordinance, sufficient study of local conditions is essential; there is danger in too hasty action. The plans and experiences of other cities should be studied, but with a knowledge of why certain methods were adopted. No direct copying is permissible.

The aim of zoning is usefulness; hence, in order to succeed, the plan must be practicable, and there must be a nice balance between what is theoretically desirable and what is practicable of accomplishment. Although many things are desirable, ideals can only be approached. The city cannot be re-made overnight. It is possible to do the best only with what is available, and look to the future for improvement.

The public must be taken into the confidence of those promoting the zoning plan. To succeed, the ordinance must be what the majority of the people want, not the desires of a few. Publicity cannot be overdone. More often than not, there is a lack of understanding in the community on the necessity or desirability for zoning regulations.

The ordinance, in general, should prescribe regulations as to the use of land and structures, the height to which buildings may be built, and the area that must be left vacant on each lot. The districts in which the different regulations are to be applied may or may not be co-terminous, or they may be combined. That arrangement which will secure the greatest simplicity should be adopted, and this will depend largely on local conditions.

Provision should be made for amendment of the ordinance under adequate safeguard, and a Board of Adjustment should be created. The function of such a board is to review any contested decision of the administrative official, to decide border lines and exceptional cases, and to adjust the regulations in harmony with their spirit in cases where a literal but too strict interpretation would cause unnecessary and excessive hardship.

Above all, the plan, to succeed, must contain only those things which are practical of accomplishment. Under the scope of police power, the law recognizes the necessity only of those things which are beneficial to the whole. Esthetic and similar considerations are not true uses of the police power as a main basis. There must be a fair relationship between the public good to be secured and the private injury suffered. Local physical conditions, usage, opinion, and all the surrounding circumstances are important in determining what will be useful, and whether the ordinance will be upheld by the Courts.

The real test of a zoning plan will be: Whether or not it works?

DISCUSSION

WILLIAM T. LYLE,* ASSOC. M. AM. SOC. C. E. (by letter).—That city planning is one of the greatest of modern movements is demonstrated by the fact that 50% of American cities of 100 000 people and more have city plans; that city planning has become increasingly interesting to the engineer is evidenced by the general trend of developments in the last three decades.

The forerunner of modern city planning in America was the planning of the grounds, buildings, and waterways of the Chicago Exposition of 1893, which was a revelation to all those in attendance. The work at Chicago was largely responsible for the idea of "the city beautiful"; the dominant note was the esthetic, and to apply this idea was, for many years, the hope of civic zealots. Although the World's Fair provided a great impetus, it is also true that city planning would have been developed without it, for other reasons and by force of necessity. The phenomenal growth of cities is well understood by members of the Profession. At the beginning of the Civil War only 11% of the population of the United States lived in cities, whereas, in 1923, nearly 40% live in cities. This growth would have been impossible without improved facilities, railroads, rapid transit, the automobile, and the telephone; without them, modern manufacturing would have been impossible. Growth and complexity of development demand a city plan; the many evils attendant on an unregulated growth demonstrate its need. The great civic awakening of the last few years has disclosed many needs and opportunities. Business demands a city plan. Adequate transportation facilities must be provided. Conflicting railroad interests must be harmonized for the business welfare of the community. Grade crossings must be eliminated. Convenient and commodious wharfage facilities must be provided, with channels and harbors of sufficient depth for shipping. Building construction must be made to conform with modern hygienic requirements concerning light, air, and congestion. Throttled thoroughfares must be cleared and widened, boulevards provided for pleasure driving, service roads for trucking, and arteries of travel located along lines of least resistance. Parks and playgrounds must be established to satisfy the recreational and artistic requirements of the community.

The year 1909 is a memorable one in the history of the development of city planning in the United States, because in that year the First National Conference on City Planning was held. It is also memorable in England by the passage of the Town Planning Act. Since 1909, city planning has advanced rapidly and has established itself as a well recognized and highly important art; and this advancement has occurred notwithstanding the retarding influence of the World War.

Many attempts have been made to define this advancement. To make a composite of all such attempts would be impossible, so different and conflicting have been the conceptions of the framers. This advancement has been due not only to the fact that the problem has been studied from different standpoints—from that of the landscape architect, the economist, the sociolo-

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gist, the city administrator, and the municipal engineer—but also because great progress has recently been made in the art itself through the making of concrete plans. In view of the increasing importance of the engineering aspect of city planning, the writer would define it as the art of systematically providing for the physical growth of a city and its environs. More particularly, it is concerned with the engineering problems of streets, parks, public buildings, transportation by water and rail, and the regulation of the use, height, and size of buildings.

City planning will continue to recognize esthetic requirements, but the problem has become so great that other elements, such as the utilitarian, the hygienic, and the economic, are now considered of greater importance. The emphasis has shifted, and the idea has grown by accretion. Modern city planning can best be characterized as an art comprehensive. It is concerned with interlocking interests which constitute a problem of the greatest intricacy, requiring a governing mind of highly developed visualizing power and coordinating ability. There are many matters in city planning concerning which the engineer is unable to speak with authority, such as legislation, administration, and esthetics, but, in the main, the writer believes that in each of its three phases—design, construction, and management—the problem is one of engineering.

The practical divisions of the subject of city planning have been presented by recognized authorities. Without attempting to discuss any one of these divisions at length, the writer will mention a few observations on each of them from his professional practice.

The park and parkway problem is assuming more and more an engineering aspect. The park problem of to-day is different from that of a few decades ago. It now constitutes an integral part of the comprehensive city plan. As the other factors in the problem—streets, railroads, and waterways—have a direct bearing on the park problem, so it, in turn, has a direct bearing on them. The great American municipalities were wisely guided when acquiring lands for park purposes. These lands were acquired while they could be had, before other operations and improvements and the rising cost of real estate rendered their purchase impossible. Fortunately, much of this land has been left undeveloped. Other tracts have been improved, often at great expense, and converted into parks of formal or informal architecture, which are the pride of the municipalities which own them. The time has come, however, when many of these parks must be rebuilt in order that they may become increasingly beneficial to the public; in order that they may become useful in a maximum degree to all the population which they are intended to serve. It is obvious that hygienic, economic, and recreational desiderata will displace the esthetic to a considerable extent.

Regional planning is another evidence of the expanding scope of civil engineering. No city is sufficient unto itself; it is dependent for its existence and for its growth on the surrounding country and on other adjacent cities with which it transacts or should transact business. A regional planning which brings contact between producer and consumer and thus stimulates both

production and consumption is a new undertaking, but one which will attain immense popularity. In regional planning, the engineer will evidently occupy a prominent position, concerned as it is bound to be with considerations of water, railway, and highway transportation. He will also be called on to solve problems of water supply and sewage disposal as joint projects, to the great saving of each co-operating municipality.

Zoning is closely related to the major street and boulevard plan as functionally considered, and to the plan for parks and playgrounds. It permits the separation of different kinds of street traffic, and thus powerfully operates in the solution of the traffic problem. The chief cause of street congestion is the promiscuous intermingling of fast and slow vehicles. This is largely reduced through the operation of the zoning system. As the streets and arteries of travel are the principal concern in city planning, zoning is very important. One of the chief objects of zoning is to stabilize real estate values, not only in the residential section, but also in the commercial and industrial sections. Regulative zoning is of such a nature as to offer the necessary facilities to business and manufacturing, such as transportation, pavements, sewers, and power. A sound zoning should be constructive rather than destructive; instead of merely operating to drive out objectionable plants and occupations, it functions rather in preventing their establishment in improper locations, and thus, indirectly, offers the facilities already mentioned. It is evident that a thorough understanding of the zoning problem necessarily requires an understanding of the nature of industry and transportation, and thus comes directly within the province of the engineer.

E. M. Walker,* Assoc. M. Am. Soc. C. E.—City planning, as usually done, does not cost much, but if it costs anything, it should be justified. It is fair to criticize some plans that have been made because they served no purpose, and resulted in little tangible progress. Their cost, however small, may seem to have been wasted, but is it not true, in commercial and other private activities, that many plans and schemes are devised only to be discarded? It is sometimes worth a great deal to know what not to do. The final developments may include elements of many of the discarded plans. Engineers plan so that when they execute they may proceed in an orderly and economic, rather than in a haphazard, expensive, and "anarchistic", manner. No one can doubt that the population of the United States will continue to increase, and will require many things that do not now exist. Regional planning and zoning are to provide that these things shall be in proper relation to each other. The cost of such planning is small in comparison with the great losses caused by haphazard development. The correction of the mistakes of the past may appear to be costly, but even these can often be shown to be worth far more than their cost. The beneficiaries, whoever they may be, should carry the burden. The usual assessment method, or the excess condemnation method, will do so if honestly and efficiently carried out. Whether or not the things shown on the plans are required should be referred to the beneficiaries for their decision.

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The people should say whether they want such things as thoroughfares, parks, playgrounds, and public buildings specifically, and, in general, a community in which living will be worth what it costs, and not cost more than it is worth. Proper city planning cannot be accused of extravagance; but to obtain proper planning requires an enlightened public opinion and "eternal vigilance."

CHARLES W. LEAVITT,* M. AM. SOC. C. E.—Mr. Lewis has featured the objects of the Committee on the Plan of New York and Its Environs, of which Committee Mr. Charles D. Norton is Chairman. It seems most fitting that Mr. Lewis should be connected with this Committee because, for many years, he has taken such a prominent part in the city's accomplishment along these lines, and is known to have her best interests generously at heart. It is highly gratifying to know that he is going on with this work, using his intimate knowledge of the conditions which obtain here; he will be able to guide the efforts of this Committee to success in their greatly needed undertaking.

Mr. Lewis states that the Sage Foundation which is helping to finance Mr. Norton's Committee, was created largely, if not wholly, for the improvement of social relationships and better living conditions among the people in New York City. The greatest difficulty in the work of engineers who have devoted years to solving city planning problems is that, in helping with the enormous undertakings of routine, they often lose sight of the extremely important fundamental principles which are the cause of the work. They find themselves so involved in the tremendous detail of the betterment of environment that they temporarily ignore the primary matter, which is the simple fact that actual living in that environment must be bettered, or their effort is futile.

Regional planning, so-termed, may be a new topic to some engineers who have not been following the more recent developments. Many cities in England have found it wise to endeavor to scatter population, rather than concentrate it at certain points. The results of concentrated population in New York and in some other cities in the United States are now seen, and it is realized how much better it would be for the lives of the people if they were not subjected to such conditions as exist, for instance, at the corner of Fifth Avenue and Forty-second Street, New York City.

In planning these districts, or regions, the idea is to make them as far as possible independent and self-contained, so that they may function in themselves and thus avoid much interurban traffic. This seems to be logical, and has the further advantage of giving the employee the opportunity of living somewhere near his work, possibly walking to it rather than traveling in a crowded carrier. Regional planning very happily might include the distribution of park areas between the various regions, so that children may get to the parks without traversing extensive built-up areas.

The questions of water supply, sewage disposal, and other municipal services may be carried on much as they are now, by laying mains of the proper sizes between the regions; or it may be advantageous to provide separate plants

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for each region. This is a matter to be determined for each individual case as it comes up for consideration.

One of the greatest arguments in favor of regional planning is that it provides greater opportunity for character development than is found in congested centers; there is developed more individual interest in the neighborhood, more pride in appearance, and a feeling that the individual voice carries weight and can obtain results; the individual citizen is not last in "the maelstrom", as he is in the center of large cities.

In the United States, property values have been rising and falling in a way to make investments in realty speculative. Realty values on Broadway, below Fourteenth Street, New York City, have gone up, then down, and are again advancing. The same holds true of the Twenty-third Street District. At present, great wealth is being poured into the Forty-second Street District. If the congestion which now threatens grows materially worse, these values may fall. It is similar to any other speculation, in that, when a certain point is passed, there is a danger zone, and the investor must be careful. Such concentration of values can be ameliorated, to some extent, by regional planning, which should appeal to the investor, even if matters of health, right living, and esthetic development do not arrest his attention.

Mr. Childs who took active part in the work of the Committee on City and Suburban Planning of the New York State Association of Real Estate Boards, helped greatly in the work done by that Committee in 1922, when an ordinance was framed making it necessary for either residents or landowners of a community to file with their local government any and all plans for the development of property, for the purpose of providing that arterial ways, parkways, and other public passages should not be stopped by private, arbitrary development, planned without consideration of the good of the whole community, whether such community be considered as village, city, county, State, or Nation. An ordinance on such a basis means much for the future. It is acknowledged that developments must be controlled by legal authority, or they will form obstructions. Through every section there should be an unobstructed thoroughfare for travel, particularly in these days when competition makes it necessary for the individual to transport himself and his stock in trade from one place to another in the shortest time possible.

The ordinance, which was in the form of a resolution when passed by the New York State Association of Real Estate Boards, is now being framed as a bill and, in due course, will be presented by the Association to proper legislative bodies for action.

As horses have disappeared to a great extent, with the exception of their continued use on bridle paths, etc., some changes must be made in the design of highways, parks, etc., to accommodate the motor-driven vehicles which take their place. There should be no grade crossings where it is possible to avoid them. The traveled way should be treated more or less as a railway, as the speed has been advanced to 10, 20, and 30 miles an hour. The occupants of cars can observe only a small part of the landscape, and the scale of park treatment, lawns, planting, fountains, statues, and other features must be on a

larger and coarser scale, or the whole picture will appear as a blur, rather than as a reality; the areas taken for development must be large, so that objects to be observed can be placed in proper perspective.

It must be remembered that the automobile has made it necessary for the modern man to live his life at least four times faster than was customary before motor cars appeared. He dashes from place to place, crowds four days' activity into one, and then wonders why he feels tired and confused. The parkway cannot escape; it must be put into scale and viewed as a moving picture film.

It is not advocated herein that landscape design should suffer by the production of plans in coarser detail, but the scale is becoming colossal, as may be observed almost anywhere throughout the country, because of the necessity of being in harmony with the wide and ever changing vistas viewed from quickly moving cars. For instance, some gate piers were designed on which, at close range, the sculptured details were charming and complete. To the pedestrian passing by, the charm still remained, but to the occupant of the automobile, going at the rate of even 15 miles an hour, it was blurred, and high relief had to be used in order to bring out the design satisfactorily.

A story is told of a traveler in England who started his automobile trip at the rate of 25 or 30 miles an hour. After traveling the first day, he attempted to record what he had seen, but soon discovered that he had seen a great deal and observed very little. Subsequent experiment proved that satisfactory observation could be made at a speed not exceeding 15 miles an hour. If the enjoyment of design is to be for automobile passengers traveling at their present rate of speed, design will have to be less intricate and more clearly defined than is now usual, or no enjoyment in it will be possible.

Where adults and children wish to walk and play, automobile drives are not now desirable. Such drives should be fenced off and protected, like railways, if they must exist. They should be made distinct from the parks which are places of rest and relaxation, with playgrounds for little children.

It may be extreme to advocate fencing off automobile ways, but the number of children who run from pedestrian paths into such ways, and are killed there, is appalling. Drastic action should be taken to prevent this useless loss of life.

Arterial ways must be provided from cities into the country, and must make proper connections between cities. As the automobile truck has developed into one of the chief methods of transportation of freight (the comparison is: Waterways, 250 000 000 tons; railroads, 2 600 000 000 tons; trucks, 1 400 000 000 tons*), and has caused congestion and danger on the highways, it may be well to provide automobile truckways separate and distinct from the ways for other travel, exclude trucks from parkways, and, as far as possible, have the automobile truckways skirt rather than pass through—but with branch connections into—the towns and cities. Parkways, on the other hand, may very properly pass through the center of the town or city, and be made part of the interurban traffic route. It is remarkable to note the growth of freight

* From address by Julius H. Barnes, Esq., President of the Chamber of Commerce of the United States, *Proceedings*, Am. Soc. C. E., for March, 1923, p. 132.

transportation by truck, and realize that it amounts to almost five times that by waterway, and more than half of that by rail.

Realty values along motorways have not advanced; such ways are not pleasant places of residence, although they are desirable near-by, for transportation purposes. It is doubtful if, at this time, there is available sufficient data to warrant the formation of an opinion as to just what is the reaction of automobile ways on adjoining realty values. On the Concourse, through the Borough of the Bronx, New York City, there has been a great deal of building. Whether or not residence there will be attractive permanently is a question, and, on the answer to this and similar questions, depends the future of automobile parkways in and through congested districts.

Although the speaker does not intend to discuss zoning, he wishes to call attention to one fundamental remark made by Mr. Knowles: "All zoning controls population". It certainly does. Whether that control is for right or for wrong remains to be seen. A zoning ordinance should be so broad as to permit of flexibility.

In the early years of the United States, the settlers had perfect zoning; they had plenty of room around the village green and they developed fine grounds which enabled them to have plenty of space where they could raise the right kind of children to populate the land. Whether or not it is right to permit the sort of building that is going on to-day around Forty-second Street, New York City, and whether or not it is right to restrict residence districts to a limited number of families per acre, are very grave questions which only time will settle. It is gratifying, however, to hear as fine an authority as Mr. Knowles state that the number of people per acre should be kept down, and that no ordinances should be passed, which cannot be regulated later, because people may find such ordinances very irritating, whether for or against them.

Mr. Knowles' paper on the relationship between the engineer and the city plan is most interesting and to the point, but it does not bring out quite strongly enough the human element which has been developed very recently in the engineer. Although the engineer is necessary for city planning, city planning has greatly benefited the engineer; nothing has ever brought out his human side like present-day city planning. The city engineer and the practicing engineer have awakened to the responsibility of their standing in the world; they find that they must appear before political bodies, meet where other people do, and must have at heart the best interests of human relationship, in order to make plans which will provide properly for right living. It has been a perfectly splendid incentive to the engineer, and has spurred him on to give to the public, in the best possible way, the benefit of his trained mind.

It is suggested that a city engineer should have very close acquaintance, always, with the city engineers in the towns nearest him; he should cultivate their acquaintance, know them, be fond of them, and endeavor to bring about proper connections between his city and theirs, thus making possible the closest suburban communication throughout the country.

FREDERIC A. DELANO,* ESQ.—The notable fact brought out by the discussion of city planning is the wide and general interest which obtains. Time was when this subject was confined to a specialized branch of the Engineering Profession, that of the military engineer. Now, however, it interests not only the engineer, but every man concerned in civic development, not solely the architect and the landscape gardener, but the sociologist; and it appears that all public-spirited citizens are actively aroused and regard the subject as one of major importance concerning everybody.

The speaker's interest was first directed to the problem in Chicago, and has been a subject of continuing interest. The Committee of the Russell Sage Foundation, referred to by Mr. Lewis, is interested in the subject in New York City, but realizes the great difficulties to be overcome and does not believe that any one man or even a small group could be found to prepare a plan for New York City and its environs which would meet with general acceptance. In an area of such diversity of conditions and interests, therefore, it would seem wiser to bring together the consensus of views and co-ordinate them if possible into a comprehensive whole.

In Chicago, of course, the problem was less difficult, and Mr. Burnham, by reason of his achievement with the World's Fair of 1893, had the confidence of the community to such a marked extent that he was readily accorded the lead, but even he saw the necessity of arousing the interest of a considerable group of business men. He began by holding meetings twice a week in his "sky parlor" office overlooking the Lake, and these meetings grew from a small group of perhaps half a dozen to four times that number. Finally, when the Plan of Chicago was made public and submitted to the City authorities, after two years of zealous work, it represented the awakened interest of at least one hundred of the leading business men of the city.

The next step was to secure general co-operation and adoption, and this was done by the selection and appointment by the Mayor of the city of a committee of about three hundred men representing every ward and section of the city. This committee, with such changes and additions as time has necessitated, is still, after 14 years, the guiding force behind the plan; but even this strong committee could not have accomplished much unaided, and the most effective assistance was secured by interesting the school children. This was accomplished by the preparation of a textbook on planning, which illustrated the work being done in other cities and that contemplated in Chicago. This booklet, put into the high school curriculum as part of a course on civics, awakened wide interest which was stimulated by prizes and commencement oratory.

The advantage of the plan was that it not only educated the coming voters, but through them did much to arouse interest among their parents and friends. This experience serves to illustrate the importance of a wide dissemination of knowledge in order to secure the co-operation of a large community.

CHARLES N. LOWRIE,† ESQ.—Changes in park design made necessary by automobile travel are extensive and interesting. They relate principally to

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† Landscape Archt., New York City.

roadways, grading, and plantations. Park playgrounds which are neighborhood conveniences, and small parks or squares which are usually in the central sections of a city, do not affect a discussion of the problem; but as soon as larger areas are considered, such as Central Park, in New York City, Prospect Park, in Brooklyn, N. Y., Franklin Park, in Boston, Mass., or others of that type generally throughout the country, the problem of automobile traffic immediately arises.

The basis of the change brought about by motoring is related primarily to the question of speed. Comparatively narrow roadways and park driveways were used with horse-drawn vehicles. Grades could be fairly steep, if necessary, and there could be reasonably sharp turns and curves. Narrow parkways, steep grades, and sharp curves, are equally difficult for motorist and pedestrian. In future parks of moderate size, it would seem desirable to minimize motor driveways as much as possible, and, in some, to transfer automobile traffic entirely to parkways or boulevards.

The question of grades, and especially curves, and the danger to pedestrians, are obvious, and call for general reduction. If grades are steep, the noise is prejudicial to the quiet and reasonable peace of a park. If heavier cuts and fills are necessary in order to obtain better grades and curves, a disarrangement of natural conditions is produced, hence care should be taken to select as secluded a line as other conditions will permit. Sharp intersections should be minimized.

As to new plantations, there is again the question of speed as a governing factor. With horse-drawn vehicles, one had sufficient time to observe landscape effects with more leisure and in more detail. At present, it is better to design the landscape scenery, as far as possible, on very broad lines, including the planting. The beauty of intimate detail is lost to the motoring public.

If fewer motor driveways are placed in the parks, more parking places should be provided at the entrances, or, if in the parks, at points where large groups are likely to assemble, such as a water-front feature, near a music pavilion, etc.

The speaker believes that the use of automobiles will do much toward linking isolated park areas into park systems by utilizing boulevards and parkways, with the parks abutting as much as possible and in some cases being elongated into a combination of parkway and park. By that process, the people who live in the immediate vicinity as well as the motoring public will have enjoyment of such parkways. Parkways are usually in the form of a circuit system, which is the most accessible type.

M. W. Weir,* M. Am. Soc. C. E.—Papers on City Planning and Regional Planning nearly always lay particular stress on the neglect of communities to plan ahead so that their physical growth may be controlled and orderly. It is the speaker's belief that education in this subject should be given to the masses in such a way as to teach them clearly the penalties to be expected if neglect of advance planning is continued persistently, and to show how vitally business interests and tax rates are affected thereby.

* Cons. Engr., New York City.

Although experience is an able teacher, her fees are high. Especially is this true when her repeated lessons are allowed to pass unnoticed by the community at large and are realized only by a few public-spirited citizens who are willing to give their time to striving for co-ordinated action by the citizens of the community.

New York City has passed through a number of experiences which by proper advance planning might have been eliminated entirely or, at least, might have been made much easier. Although New York City has made plans for her future, she even now is confronted with new problems which, in a way, are an outgrowth of the very plans already made, because those plans did not go into the possibilities of the future with sufficient detail, particularly in regard to transit facilities and the ultimate limits thereof.

At the moment, additional lines of transportation into centers of business activity in New York City without doubt will relieve congestion of transit leading to those centers, but, at the same time, they cannot but increase the congested condition of the streets in those centers. It is obvious that increasing the street areas to any appreciable extent is out of the question, because of prohibitive expense and the consequent loss of area for building use. Such increase, if carried far enough, naturally would cause a total failure of purpose. Although New York City furnishes a good example of conditions resulting from years of previously unrestricted growth, many other cities manifestly are approaching like problems of no small magnitude, and if prompt action is not taken, some will be fit, if lamentable, illustrations of the lesson at hand. It has been said that it is advisable and necessary to by-pass through street traffic away from business centers. This attitude, although correct under ideal conditions, is in some respects open to objection, from the viewpoint of practicality. When the history of retail business is examined, it will be found that business, in search for success, has found satisfactory location on arteries of heavy traffic, in fact, it is now the practice of a certain chain of retail stores to determine their locations on the basis of the traffic passing the places under discussion.

An example of some value in illustrating this point occurred not long ago at Port Chester, N. Y., where the Boston Post Road—a through road of heavy and diversified traffic—becomes the main thoroughfare of the business district of the town. Because of temporary necessities of construction, for a time the through traffic was diverted to another street one block north. Almost immediately merchants on the Boston Post Road complained of loss of business and, as time passed and the idea of making the diversion permanent was suggested, they protested bitterly, stating that as much as 40% of their business in some lines was transient and derived from the through traffic on the Road.

To illustrate further the relation between successful business enterprise and traffic: There are points where transportation facilities draw large numbers of people, such as the Summit Avenue Station of the Hudson Tubes, and several points in the Bronx at or near transit stations. A few years ago, these points were residential only, but since transportation facilities have

increased the street traffic, important business districts have grown up and become ambitious to gain greater importance. Communities, as well as individuals, have ambitions; they have the need and desire to expand in order to gain in proportions and importance. This is only natural and healthy; obviously, it is folly to attempt to atrophy such growth or to throttle such ambition.

City Planning and Regional Planning, to be efficacious and successful, must show proper regard for the principles of political economy which, of necessity, enter into the problem, and for the natural desires and ambitions of the people comprising the community affected.

E. A. FISHER,* M. AM. SOC. C. E.—The City Planning Bureau of Rochester, N. Y., has now been in operation for about 4½ years. The city has no City Planning Commission, as many other cities have. Its city planning is done by a Bureau in the Department of Engineering. One great advantage of this is that this Department is utilized in making the necessary surveys and maps, and it also puts the Department in close touch with what the Bureau is doing.

This Bureau is composed of two parts, a Superintendent of City Planning and an Advisory Board composed of four citizens and the Corporation Counsel. There are no commissions in Rochester; all operations are under the single-headed department plan; thus the Superintendent takes the place of a commissioner.

The city has been able to secure the services of some of the most prominent and public-spirited citizens, those having knowledge of the needs of the city, as members of this Board. A former Mayor served as its Chairman for four years. The action of the Superintendent is presented to this Board, and, when approved, is final. Members of the Art Commission are serving as members of the Advisory Board.

As the duties of the Superintendent, as far as they are related to the laying out of streets and matters of that kind, are similar to those of a commissioner, it is not necessary to describe them.

With reference to zoning, the code provides only for zoning for "use". The experience in Rochester in relation to zoning was altogether different from that of Detroit. Public hearings were started in order to educate the public, as had been done in other places, but this did not meet with success, as very few people would attend. The matter was presented to the different civic organizations, and the general consensus of opinion was that the zoning should be carried on, and should have been done years before. It was concluded, therefore, to adopt zoning regulations. It was not necessary to have an ordinance, as is required under the General City Planning Act. The Superintendent made up the regulations for "use" zoning, presented them to the Advisory Board, and on September 22, 1919, they were adopted, put in operation, and have done a great deal of good. Since the zoning was put into effect, quite a number of people have found out what it means.

The Superintendent has stated publicly, as his belief, that any system of zoning that cannot be changed without too much trouble is worse than none.

* Cons. Engr. and Supt. City Planning Bureau, Rochester, N. Y.

There has been no general change in the zoning plan, but numerous small changes have been made. It has been necessary also to make exceptions. It was found in New York City that, in order to prevent undue hardship, it was absolutely necessary to create some board that would have the power to deviate from the strict regulations of the zoning ordinance, and the Board of Standards and Appeals was permitted or authorized to allow special variations.

A Board of Appeals was not needed in Rochester, because it was considered that the Advisory Board had the authority and could act as such a Board.

A recent decision relative to an exception made by the New York Board of Standards and Appeals, is of much interest, because the same thing is done in Rochester, and, if the city did not have authority to do it, it would result in a great deal of harm.

The City authorities also believe in the employment of experts who have had experience in other cities, and do not believe in turning over the plan to experts to make up a plan and carry it out, as that belongs to the city itself. The city has employed quite a number of experts, eminent in special lines of work.

As the Common Council considered the height and area of buildings as a part of the Building Code, an amendment to the charter was secured which provided that the Common Council might enact an ordinance governing such matters.

The plan was prepared by the Superintendent of City Planning, and a first ordinance has been adopted. The Act contained a special provision that no ordinance could be passed without the approval of the City Planning Bureau, so that "height and area" are considered with the "use".

This covers in a general way what is being done in Rochester. The ordinance has been in operation about 4½ years, and there have been no cases in the Courts. There has been considerable individual criticism, but the general feeling in the city is in favor of carrying out this plan.

RUDOLPH HERING,* M. AM. SOC. C. E.—With reference to the papers under discussion, the speaker would like to add a few historical remarks.

In 1887, the speaker was Chief Engineer of the Drainage and Water Supply Commission, in Chicago, Ill. While in that city he joined the Chicago Literary Club and was asked to read a paper before it. Not wishing to present a purely engineering subject, and having made some recent studies regarding the natural development of cities, he selected a subject which he thought might be of some interest to the citizens of a large and modern city, especially one of rapid growth.

The theory of evolution in general, and particularly with reference to social conditions, had always interested him, and among others the natural growth of centers of population seemed to follow laws the recognition of which could be helpful in understanding and furthering development. He had collected, where available, the plans of old cities in Europe and Asia at different

* Cons. Engr., New York City. Mr. Hering died on May 30, 1923.

stages of their growth. A set of plans of Paris from the Middle Ages to modern times was particularly interesting, and was given later to the Library of the University of Pennsylvania.

Literature on this subject had also been collected, partly from England, where the so-called "Garden Cities" had been developed more than a generation before. The best book on this subject was found to be one compiled, perhaps 40 years ago, by the City Engineer of Cologne, Mr. Stübben, who was later transferred to Berlin, and became responsible for some of the extensions of that city.

All this material helped the speaker to trace what seemed to be laws which the natural needs of cities indicated, and which appeared, at first automatically and then through engineers, to be utilized with benefit and economy to the citizens.

Much of the matter referred to in these papers was recognized or indicated in Europe a generation ago. Mr. Stübben speaks of the regional development and of zoning with the differentiation of occupations. The speaker referred also to the natural zoning that had automatically developed in New York City, because it was helpful to business, and pointed out the centers for banking, for wholesaling, for dry goods, jewelry, hardware, for amusements, for high, medium, and low-class residences, etc., already well differentiated at that time. L. M. Haupt, M. Am. Soc. C. E., in the Seventies, pointed out, before the Engineers' Club of Philadelphia, the advantages of street layouts with reference to economy of travel, advocating diagonal avenues to connect the various centers of business and residence, with reference to the saving of time, labor, and wear.

Such were the chief data for the Chicago paper. It received the usual polite applause, but the speaker felt that the subject was not yet fully appreciated, even in a city like Chicago, at a time when such a study might have been of great economic value. Of those present, only one gentleman, Mr. Burnham, the mention of whose name by Mr. Delano called forth these remarks by the speaker, made a few comments. In effect, he said: "That is all very well, but is beyond the appreciation of the present city officials and property owners." As architect of the World's Fair buildings and grounds in Chicago in 1893, Mr. Burnham himself showed in his magnificent designs and suggestions that he was already alive to the new movement in Europe. Since that time, it has been quite naturally accepted also in the United States, and pursued by a number of able American engineers with, the speaker hopes, great advantage to the future development of the cities of this country.

HAROLD M. LEWIS,* M. AM. SOC. C. E.—In his paper on "Regional Planning", Mr. Nelson P. Lewis has referred to the desirability of establishing a system of zones, or rings, of open spaces, which might help to prevent the enormous increase of congestion in metropolitan areas. Such spaces would necessarily bear a vital relation to the park and boulevard system, and one can readily imagine the whole area still further divided into sectors by a system of

* Executive Engr., Russell Sage Foundation, New York City.

radiating boulevards. Two possible illustrations of this theory may be mentioned.

New Jersey has recently received a wonderful opportunity for establishing what might be called a part of the framework of such a system. Agreements have recently been completed whereby the Morris Canal, which was chartered in 1824 for 99 years, will be acquired by the State of New Jersey which will thereby come into possession of a strip of territory about 90 miles long and averaging about 79 ft. in width. It illustrates the possibility that such a strip would have in breaking up the congested areas through which it passes, and the congestion that will undoubtedly develop along those parts of it which are now on the edges of the metropolis. At the same time, it portrays the difficulties of obtaining regional co-operation. Lately, many suggestions have been made for different uses of this land by the municipalities along its route for their own local projects, but it would be a shame if it could not be developed in some manner so as to give the advantages of its use to all the public in that part of New Jersey, as well as in the other parts of the metropolitan area of New York.

The second illustration has developed from a study of the facilities for recreation which has become increasingly important in the last few years as the stress and strain of modern business has increased. Table 2, compiled by the Physical Survey of the Plan of New York, is submitted in order to show the relation, or perhaps discrepancy, between those areas in the suburbs of New York City now available for public recreation and those used for private recreation, as shown by the areas of golf and country clubs within the district. The figures have been computed for the public parks and the golf and country clubs in Nassau, Suffolk, and Westchester Counties in New York State, and also that part of Fairfield County, Connecticut, which might be included in New York and its environs.

TABLE 2.

	Areas, in Acres.		Percentage of Total Area.		Assessed valuation of golf and country clubs.	Ratio of club areas to park areas.
	Public parks.	Golf and country clubs.	Public parks.	Golf and country clubs.		
Nassau County, N. Y.	70	3 400	0.04	1.9	$2 342 000	50
Suffolk County, N. Y.	76	3 000	0.01	0.5	1 518 000	40
Westchester County, N. Y.	2 288	5 230	0.41	1.8	10 598 000	2.3
Part of Fairfield County, Conn.	714	1 360	0.27	0.5	804 000	1.9
Totals	3 148	12 990	0.21	0.86	$15 262 000	4.1

Deducting from the figures for Westchester County, the Bronx River Parkway which is under the control of a State-appointed Commission, and the recently acquired Mohansic Park, Westchester would have only 288 acres of parks, and its ratio of club areas to park areas becomes 18, and the total ratio

becomes 11. Parks of less than 1 acre in area have not been considered in Table 2.

The speaker believes these figures show, in a striking way, two things, namely, the lack of park provision in these parts of the Metropolitan Area and the great demand for recreation facilities. These club areas are now important but temporary open spaces, and the suggestion is not a new one that they might readily be made permanent if some public authority, or commission, would acquire these parcels and then rent them back to the clubs for such a number of years as would correspond to the period that they would naturally stay there in any case. All these clubs are more or less temporary in character, and tend to move farther out from the city as congestion develops and the value of land increases. The speaker believes this possibility would be well worth study, not only in the New York Metropolitan Area, but wherever there is danger of over-congestion.

GEORGE P. HEMSTREET,* ESQ.—Regional planning is a splendid idea. Proper zoning undoubtedly adds to the value of the community, but, having made the plans for improving the outlying Metropolitan District, who is going to pay the bill?

The natural answer is that the tax payer will make the investment, but that the increased value of his property due to the improvements will repay him amply for the money expended.

The rent payer looks at improvements from this angle: Improvements cost money. Result: Increased taxes. Final result: Increased rents.

In cities of the first class, the rent payer has some protection from the unscrupulous landlord, due to the emergency laws governing rentals. These laws do not apply to villages adjoining cities of the first class, and as a result rents have been raised to such an extent in the villages of the Metropolitan Area, that people of modest means are becoming desperate.

Under the school laws, rent payers as well as property owners can vote on a proposition to raise school funds through a bond issue, and, in numerous instances, appropriations for increased school facilities have been defeated by the vote of the rent payer who has been willing that his children should go on part time or be herded together in an unsanitary manner, rather than have the rents raised again.

Mr. Lewis has mentioned the fact that in the Metropolitan District there are nearly 400 different municipalities. Regional planning in the Metropolitan District covering these numerous municipalities would undoubtedly meet with violent opposition from the rent payer when the time came to vote the money to carry out such needed improvements. These are hard, cold, disagreeable facts, and they must be faced.

Before additional funds can be voted for parks, sewer systems, broad avenues, splendid public buildings, and all those public things that add so much to the value of the community, it will be necessary to satisfy the rent payer that all these needed improvements are not going to be loaded on him, together with a handsome profit for his landlord.

* Hastings-on-Hudson, N. Y.

GEORGE A. SOPER,* M. AM. SOC. C. E.—The speaker would like to have one of those who have spoken for the Sage Committee on The New York Plan explain just what is meant by the use of the word "plan"? Some who have heard of the New York Plan seem to think that a scheme of physical development is contemplated, such as might be represented on a map or series of maps. Some think that the word "plan" is used to mean a scheme of development, together with a statement of policies and procedures which should be followed in order to carry the physical plan into effect.

The speaker is sure that all are much impressed by the thoroughness with which the fundamental data necessary to comprehensive planning in the Metropolitan territory are being collected, but some do not quite understand what tangible form the Committee's conclusions are to take. It is hoped that it is not premature for the Committee to explain this matter.

Is the plan to be announced as a definite and complete thing at some future time, or is it to be a continuing project, which will be revised, extended, and developed for an indefinite period in the future, announcement of certain features being made occasionally as seems suitable?

There are many in the Metropolitan District who would like to be enlightened on these two questions, if it is not inconsistent with the policy of the Committee to make its purpose clear with respect thereto.

HAROLD A. CAPARN,† ESQ.—The general problem of city planning resolves itself into one of transportation of man and his commerce. Even zoning is a part of this problem, because one of its purposes is to regulate transportation.

In New York City, the question of city planning seems to be one of building additional subways, so that it may be possible for more people to travel farther and farther from their homes to their business and back every day. This kind of program, however, cannot go on indefinitely. It merely results in making the city larger, more unwieldly, and more difficult in which to live and do business. Yet, like other large cities, New York takes a pride in her size, as Mr. Lewis had pointed out, which seems to the speaker somewhat like a man 5½ ft. high taking pride in weighing 600 pounds. The solution should lie not in multiplying transportation, but in making it unnecessary, in studying how subways can be dispensed with, so that most people might be able to live within walking distance of their work. The speaker does not know how this could be done, but thinks that every city planner should work with this idea in mind.

Another subject that should be discussed is the condition of the city parks. Parks are a very important part of the city plan, but would fail in many of their best functions if they were not maintained properly. Many of the parks in New York City are in unsatisfactory condition, and people are wondering why. The fault is not with the Commissioners, or the Mayor who appoints them, but in the method of administration, as this differs from that of any other park system with which the speaker is acquainted. Five Commissioners

* Cons. Engr., New York City.
† Landscape Archt., New York City.

(so-called) are appointed by the Mayor every four years. They are mostly inexperienced in park affairs, and by the time they have acquired some education at the public expense, they are dropped; then, usually, five new and inexperienced men are appointed by the incoming administration, and the same process is repeated.

If one city department more than another needs continuity of policy in its management, it is that of the Park Department. Many years are required to make and develop a park, and, in New York City it would probably take many years under the best conditions of management to ascertain how to adapt it to the best needs of the varied and changing population and conditions. The parks should be administered by men trained in park affairs and secure in office as long as they make good. This principle should prevail throughout the whole organization. "Hiring and firing" should depend on merit only, not on political pull. In fact, the parks should be removed from politics and operated like any other large business concern. Yet a continuity of policy is made practically impossible by the vicious provisions of the city charter which relate to the Park Department.

The New York Chapter of the American Society of Landscape Architects has drawn up a plan for the reform of the Park Department, which its members would be very glad to explain to this or any other body of citizens interested in city affairs.

NELSON P. LEWIS,* M. AM. SOC. C. E.—Mr. Leavitt has made the interesting observation that, as a result of the greater speed of vehicles, their occupants cannot see and appreciate landscape or other details along the highways if they are treated in much detail, everything appearing somewhat blurred, and that park treatment and other details must be on a larger and coarser scale if they are to be appreciated. He also notes that, although the engineer is necessary in city planning, his interest in the subject has done, and is doing, much to bring out his human side.

As a result of his broad experience, Mr. Delano has shown that a city planning project, if it is to become more than a plan, must have behind it an intelligent public sentiment and enthusiasm such as was developed in connection with the Chicago Plan.

Mr. Weir's remarks should not be considered as a plea for traffic congestion as a stimulus to retail business and realty values, and yet the speaker recalls instances where attempts to relieve congestion and facilitate movement have been resisted by the owners of business property as calculated to reduce its rental value.

That "regional planning" is necessary to insure better facilities for healthful recreation is clearly indicated by the statistics given by Mr. H. M. Lewis, showing the great disparity between such facilities which are available for enjoyment by the public and those open only to members of more or less exclusive organizations.

Mr. Soper has asked whether what has been called the "Plan of New York and Its Environs" is to be a concrete and complete thing at any definite

* Cons. Engr., New York City.

future time, or whether it is to be constantly revised and continued for an indefinite time. This is a difficult question to answer, as the speaker can make no statement concerning the policy of the Committee, and his active connection, as already explained, has been confined almost entirely to what has been described as the Physical Survey. To the present time, this has consisted chiefly of an inventory, or an attempt to visualize the problem. To this end, many maps and charts have been prepared, which can be seen at 130 East 22d St., New York City, and will be gladly explained to any who are sufficiently interested to call there. Although plans for a town, or a region, are sometimes referred to as complete or finished, there is no such thing. Conditions are changing constantly, and such plans can never be complete or final. They will be to a large extent suggestive only, and this is necessarily the case where hundreds of political units, in three States, are involved. There are many special lines of investigation which might be taken up and translated into definite plans by the officials of a particular city or town, but which, in the study now under way, can only be suggested as worthy of careful consideration by the local authorities.

In the speaker's paper, the need of regional planning and the attempts already made in this direction were pointed out. Such planning must be a continuous performance. No group or combination of municipal, county, or State governments, however harmoniously they might work together, could produce a complete and final regional plan. An unofficial group of citizens may be able to indicate to the local officials how the plans for adjoining political units may be made to harmonize with each other and fit into a comprehensive regional plan. If this is done tactfully, and if the suggestions are found to possess real merit, the work of the existing committee will have been justified.

Morris Knowles,* M. Am. Soc. C. E. (by letter).—In view of the paucity of the discussion, perhaps it will not be amiss to repeat the writer's remarks "it may be said that the zoning plan is primarily an engineering problem". But, is the engineer interested? The number or length of the criticisms would not seem to indicate it. However, there are some promising indications, and the suggestion to form a "City Planning Section" in the Society is one which speaks better for the future.

Professor Lyle aptly calls attention to the relation of zoning to the major street plan and that when adequately developed and by proper co-operation, congestion is thereby relieved. He refers to another fact of greatest value and always to be remembered, namely, that zoning is "constructive rather than destructive". It is not retro-active.

The discussion by Mr. Leavitt adds to the value of the paper, but there should be no doubt as to the wisdom of controlling population, that is, its location, its congestion—as long as it is done with a true sense of reliance on health, safety, and general welfare by power of the law. Again, the use of the word "regulated" is unfortunate, if by such is meant to temper or modify because of "irritation". Ordinances should be capable of amendment by the use of reasonable, but not too easy, machinery, and interpreted

* Cons. Engr.; Pres., Morris Knowles, Inc., Pittsburgh, Pa.

(by appellate boards, if necessary) with reason, so that unnecessary hardship or confiscation will not result. More than this, however, is unwise and lacking in stability.

Mr. Leavitt calls attention to the oft-forgotten factor, namely, the human equation in engineering, and, as he states, perhaps no field is calculated so well to develop this human trait as a study of the application of zoning and an endeavor to put an ordinance in effect.

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Paper No. 1527

ELASTIC STRESSES IN THE ROCK SURROUNDING PRESSURE TUNNELS

By Charles P. Dunn,* Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. B. F. Jakobsen, Jacob Feld, William Cain, J. Charles Rathbun, and Charles P. Dunn.

Synopsis

This paper is a theoretical study of the stresses occurring in the rock surrounding pressure tunnels, the purpose of which is to assist in developing pressure-tunnel design, with particular reference to that part of high-pressure power conduits and penstock tunnels in which the rock cover alone is insufficient to withstand safely the internal pressure. A method is outlined for making calculations to determine whether strength must be provided in the tunnel lining, and, if so, how much.

The subject is discussed as follows:

Part I, Theory, in which is presented a study of elastic stresses caused by internal pressure in a circular tunnel, the rock being assumed to be a homogeneous, weightless substance, with the modulus of elasticity and factor of lateral deformation as of rock.

Part II, Practical Application, in which is described a method of applying the results of Part I to tunnel design, the weight of the covering material being considered.

Introduction

This subject is of interest to many engineers because the large size of modern power installations makes the pressure tunnel the most economical form of conduit at many sites. A great number of such tunnels have been built recently, and many more are under construction or are contemplated.

The writer realizes that, on account of the hazards and uncertainties of tunnel construction, pressure-tunnel design must always be controlled more by experience and common sense than by theory, but he believes that a con-

* Designing Engr., Portland Ry., Light & Power Co., Portland, Ore.

sideration of the problem as presented in this paper will aid in forming a correct opinion.

In studying theoretically a problem of this kind, assumptions must be made, which are bound to be in error to some extent and which will not fit exactly all conditions. The results of a study may be accurate mathematically, but the practical accuracy cannot be of greater refinement than the accuracy of the basic assumptions.

The theory as presented takes no account of fault planes or stratification, the rock being assumed to have a uniform modulus of elasticity and factor of lateral deformation in all directions. For simplicity, all the calculations are based on circular bores and are, therefore, only roughly applicable to other tunnel sections.

The problem is treated as if the tunnel was a "thick cylinder" with an infinite external radius. This is sufficiently accurate for a practical study of the stresses due to internal pressure because, as will be shown, the great bulk of the stress is in the material immediately surrounding the bore, and only a small error is introduced by assuming the external radius to be infinite when actually it may be as small as 50 ft. over a tunnel 10 ft. in diameter.

I.—Theory

Consider, first, a tunnel of circular section in a weightless material having otherwise the characteristics of homogeneous granite. Assume for the moment that the rock can carry tensile stresses elastically. Later, a reduction of compressive stress will be substituted for this tensile stress, which is assumed to behave in the same manner within the elastic limit, thus bringing the solution of the problem into the field of practical usefulness.

When fluid pressure is applied to the tunnel, the diameter will increase elastically until the internal pressure is balanced, the increase being a maximum at the wall of the tunnel and zero at an infinite radius. The deformation of the rock in diametral directions must occur in such a way that all its particles are in equilibrium. The successive annular rings will increase in diameter elastically until tension is set up in them, the direct radial stresses being gradually balanced by circumferential tension, until, at an infinite radius, all the stresses in radial directions will have been absorbed or balanced by circumferential tension.

Notation.—In this paper, L, R, and T denote stresses longitudinal, radial, and tangential, with respect to the axis of the tunnel. The subscript, a, denotes apparent stress, or stress external to the particle under consideration, and the subscript, t, the true or internal stress as measured by the deformation. Additional subscripts are used to locate the stress being discussed, such as R_{ax} to denote the apparent stress in a radial direction at the radius, x.

E = the modulus of elasticity;
λ = the factor of lateral deformation (Poisson's ratio);
e = the unit elongation;
r = the radius of circular tunnel; and
x or z = the radius to any point in the rock.

Referring to Fig. 1, consider a particle between r, an annulus at the wall of the tunnel, and z, another annulus of infinitesimally greater radius

than r, the particle being enclosed between longitudinal and cross-sectional planes and the cylinders, r and z. When pressure is applied to the tunnel, there will be a certain definite elastic increase in the radius of the cylinder, r, and another (smaller) elastic increase in the radius of the cylinder, z. The condition of equilibrium requires that the tangential and radial stresses must adjust themselves so that the difference between these two increases in radius will be equivalent to the deformation caused by the mean true radial compressive stress between r and z acting through the distance, r-z, and the increase in radius at z must be the same as it would be if the cylinder, z, was the surface of a tunnel subjected to the unit pressure, R_{az}. R_{az} is less than the unit stress, R_{ar}, by the amount balanced by the circumferential tension between r and z, and it is also smaller than R_{ar} in inverse proportion to the length of the radii, r and z, because, as the particle under consideration is enclosed between radial (longitudinal) planes, the areas of its inner and outer surfaces are in proportion to the length of the respective radii.

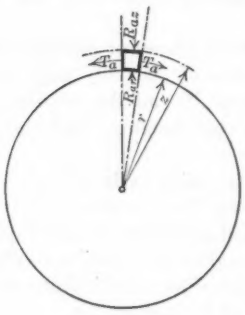


FIG. 1.

Fig. 2 shows an infinitesimal particle of rock held in equilibrium by the three pairs of equal and opposite forces, T_a, R_a, and L_a. Equations (1), (2), and (3) are based on the theory of the true stresses:*

$$R_t = R_a - \lambda L_a - \lambda T_a \quad \ldots\ldots(1)$$

$$T_t = T_a - \lambda L_a - \lambda R_a \quad \ldots\ldots(2)$$

$$L_t = L_a - \lambda T_a - \lambda R_a \quad \ldots\ldots(3)$$

It is apparent that the conditions in a tunnel are quite different from those in a gun or other finite "thick cylinder" which is free to deform longitudinally. It is assumed that there is no deformation of the rock longitudinally or parallel to the axis of the tunnel, because the tunnel may be considered very long in proportion to its diameter, and the thickness of the shell of the "thick cylinder" is very great in proportion to the diameter, therefore, the tendency toward longitudinal deformation is resisted by heavy shearing and bending stresses. It is realized that this assumption cannot be strictly correct, that there must be some deformation longitudinally, particularly near the ends of the tunnel, but the assumption of no longitudinal deformation appears to be much nearer the truth than that of absolutely free movement lengthwise as with a gun cylinder. Any intermediate consideration would introduce unwarranted

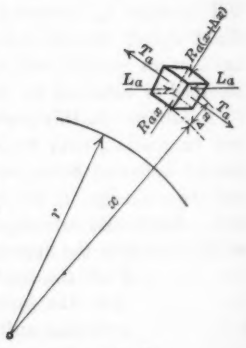


FIG. 2.

* "Mechanics of Materials", by Mansfield Merriman, M. Am. Soc. C. E., 1908 Edition, p. 360, Article 139.

complications in the problem, as the percentage of error involved in this assumption is small.

Then,

$$L_t = 0 \quad \ldots\ldots (4)$$

and from Equation (3):

$$L_a = \lambda\, T_a + \lambda\, R_a \quad \ldots\ldots (5)$$

from Equations (1) and (5):

$$R_t = (1 - \lambda^2)\, R_a - (\lambda + \lambda^2)\, T_a \quad \ldots\ldots (6)$$

from Equations (2) and (5):

$$T_t = (1 - \lambda^2)\, T_a - (\lambda + \lambda^2)\, R_a \quad \ldots\ldots (7)$$

from Equation (7):

$$T_a = \frac{T_t + (\lambda + \lambda^2)\, R_a}{(1 - \lambda^2)} \quad \ldots\ldots (8)$$

from Equation (6):

$$T_a = -\frac{R_t - (1 - \lambda^2)\, R_a}{(\lambda + \lambda^2)} \quad \ldots\ldots (9)$$

from Equation (6):

$$R_a = \frac{R_t + (\lambda + \lambda^2)\, T_a}{(1 - \lambda^2)} \quad \ldots\ldots (10)$$

from Equation (7):

$$R_a = -\frac{T_t - (1 - \lambda^2)\, T_a}{(\lambda + \lambda^2)} \quad \ldots\ldots (11)$$

also, as true unit stress is proportional to deformation:

$$T_t = e\, E \quad \ldots\ldots (12)$$

As the modulus of elasticity and Poisson's ratio are assumed to be the same in all directions, it is reasonable to consider (for stresses within the elastic limit) that,

$$\text{Deformation} = R_a \times \text{a constant}$$

or,

$$e \text{ (unit deformation)} = R_a \times \text{a constant} \quad \ldots\ldots (13)$$

then, from Equations (8) and (12):

$$T_a = \frac{e\, E + (\lambda + \lambda^2)\, R_a}{(1 - \lambda^2)} \quad \ldots\ldots (14)$$

from Equations (7) and (12), or from Equation (14):

$$e\, E = (1 - \lambda^2)\, T_a - (\lambda + \lambda^2)\, R_a \quad \ldots\ldots (15)$$

from Equations (15) and (13):

$$\frac{R_a}{T_a} = \frac{(1 - \lambda^2)}{(\lambda + \lambda^2) + E\, C} = \text{a constant} \quad \ldots\ldots (16)$$

Referring to Fig. 3, in order that equilibrium may exist, the total of all the forces due to internal pressure acting perpendicular to a radius within the bore, B, must be equal and opposite to the total of all the forces acting perpendicular to the same radius outside the bore, A.

As the external radius is considered to be infinite, the curve of T_a, starting at a definite value at the wall of the tunnel, must diminish to zero at an

infinite radius in such a manner that the area, A, between the curve and the x-axis, will be equal to the area, B.

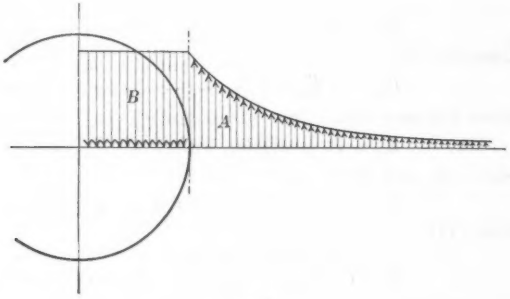


FIG. 3.

The equilibrium of forces may be stated in another manner, as shown in Fig. 4. Taking the forces perpendicular to any finite radius, x, the radial pressure on the annulus, x, is R_{ax}, and in order that equilibrium may exist,

$$R_{ax}\, x - \frac{2\, T_{ax} + \Delta\, T_{ax}}{2}\, \Delta\, x$$

must be equal to:

$$(R_{ax} - \Delta\, R_{ax})\, (x + \Delta\, x) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

which reduces to,

$$R_{ax}\, d\, x - x\, d\, R_{ax} = -\, T_{ax}\, d\, x \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

The writer finds himself unable to write the equations of the stress curves by the use of calculus. He has used "cut and try" methods based on the relations established previously.

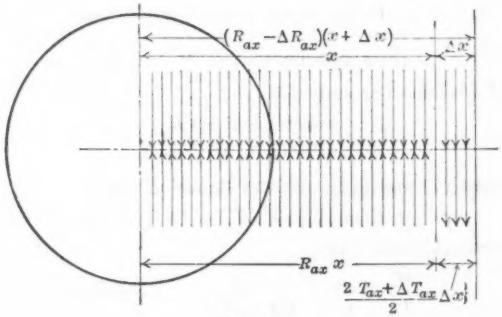


FIG. 4.

The method followed was the calculation of stress and deformation curves, as shown in Fig. 5, by assuming, first, a value for the internal pressure, R_{ar}; and, second, an estimate of the corresponding value for the elastic increase in the diameter, $2r$. The curves were determined by calculating the stresses corresponding to small increments of radius successively. Starting at the

wall of the tunnel, and choosing a certain increment in radius, the mean elastic increase in diameter and the mean radial unit stress were estimated for the increment. From these estimated values, Mean T_t was calculated by Equation (12) and Mean T_a by Equation (8).

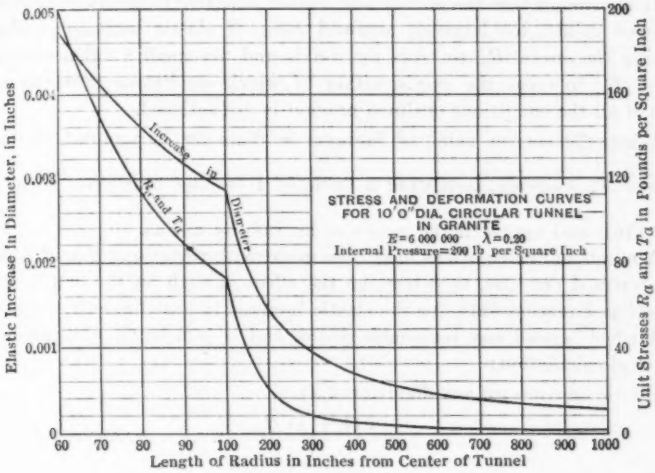


FIG. 5.

The radial unit stress at x, balanced by tension in the annulus of thickness, Δx, is, therefore, $\frac{\text{Mean } T_a \, \Delta x}{x}$, which is subtracted from R_{ax}. This remaining force, $R_{ax} - \frac{\text{Mean } T_a \, \Delta x}{x}$, is transmitted radially to the outside of the annulus to the point where the radius is $x + \Delta x$, the unit stress being reduced in direct proportion to the increase in area, or inversely as the change in radius. Therefore,

$$R_{a\,[x+\Delta x]} = \frac{x}{x + \Delta x}\left(R_{ax} - \frac{\text{Mean } T_a^* \, \Delta x}{x}\right) \ldots\ldots\ldots\ldots (19)$$

The value of $R_{a\,[x+\Delta x]}$ having thus been approximated, the estimated value of Mean R_a for the increment may be checked and corrected, and the calculation repeated if necessary. By plotting the curve step by step with the calculation, the values could be estimated quite accurately in advance.

Mean R_t was then calculated by Equation (6), and the deformation due to true radial compression in the annulus, Δx, is calculated by:

$$\text{Deformation in thickness, } \Delta x = \frac{\text{Mean } R_t\dagger \, \Delta x}{E} \ldots\ldots\ldots (20)$$

The increase of the radius, $x + \Delta x$, due to elastic stress is less than the increase of the radius, x, by the amount of compressive deformation in thick-

* Mean T_a denotes the mean between x and Δx.

† Mean R_t denotes the mean between x and Δx.

ness, Δx, in a radial direction. This latter was determined by Equation (20) and subtracted from the increase at the radius, x. The amount of the elastic increase in diameter at $x + \Delta x$ having thus been approximated, the value assumed for the mean could be checked and corrected, if necessary.

It was found that the curves were rather sensitive to errors in preliminary assumed values, too large an assumed value of elastic increase in diameter causing the curve, R_a, to cross the x-axis and too small a value causing the curve, T_a, to cross the x-axis, either of which conditions evidently cannot exist if all the conditions outlined previously are satisfied.

When the correct value of increase in diameter is assumed, the curves, R_a and T_a, coincide, indicating a value of 1 for the constant, $\frac{R_a}{T_a}$, in Equation (16), and approach the x-axis at an infinite radius.

A number of calculations have been made, with variations of λ, E, diameter, and internal pressure, to determine the effect of each on the result, and the following Equation (21), for the elastic increase in any diameter, $2x$, has been established beyond any reasonable doubt, and is sufficiently accurate for any practical calculation:

Elastic increase (of any diameter, $2x$)

$$= \frac{2(1+\lambda) R_{ar} r^2}{E x} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

Elastic increase (of diameter of tunnel, $2r$)

$$= \frac{2 r (1+\lambda) R_{ar}}{E} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

Equation (23), for the stress curves, R_a and T_a, was also established by the calculations:

$$T_{ax} = R_{ax} = \frac{r^2}{x^2} R_{ar} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

Referring to Fig. 3, this calculation is checked by,

$$R_{ar} r = \int_r^\infty R_{ax} \, d x = r^2 R_{ar} \int_r^\infty \frac{1}{x^2} \, d x \quad \ldots\ldots\ldots\ldots (24)$$

Fig. 6 shows the elastic change in diameter, in inches, of a circular tunnel in rock for various values of λ, E, diameter, and internal fluid pressure, when the external radius is infinite.

II.—Practical Application

Referring to Fig. 7, consider, first, the forces holding the rock particles in equilibrium before the tunnel is excavated.

The vertical stresses, v, are imposed by the weight of the overlying material, and the horizontal stresses, h, are imposed by the tendency to lateral deformation. It may be assumed that, in a large body of rock, lateral deformation does not actually occur, because the horizontal forces in any given horizontal plane are equal and opposite in all directions. If this assumption is made, the true

or internal horizontal stress is zero, and an apparent or external horizontal stress exists in all directions in a horizontal plane which is measured by λv.

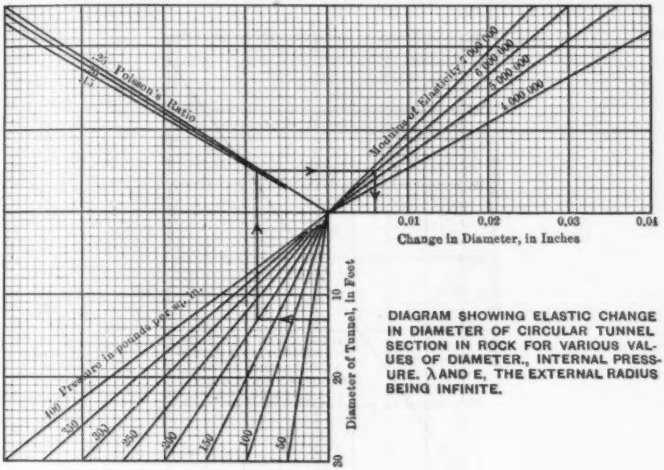


DIAGRAM SHOWING ELASTIC CHANGE IN DIAMETER OF CIRCULAR TUNNEL SECTION IN ROCK FOR VARIOUS VALUES OF DIAMETER., INTERNAL PRESSURE. λ AND E, THE EXTERNAL RADIUS BEING INFINITE.

FIG. 6.

The true vertical stress is, therefore,

$$v_t = v_a - 2\lambda^2 v_a = v_a (1 - 2\lambda^2) \ldots\ldots\ldots\ldots\ldots\ldots (25)$$

When the tunnel is excavated, it may be said that the struts, H and V, are removed, which brings about a change in stresses such that the loads formerly carried by these struts are carried by circumferential compression, and, within the elastic limit, the distribution of the additional stresses will be as presented in Part I, with opposite sign. The apparent circumferential compressive stress, T_a, at Point A, caused by the removal of the force, V, will be equal in amount to V; this will be added to the horizontal stress, H, already existing at A, making the total apparent tangential stress at A equal to $V + H$. In like manner (neglecting the difference in the quantity of material overlying A and B), the tangential stress at B is the same as that at A, namely, $H + V$.

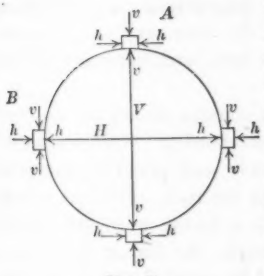


FIG. 7.

If internal fluid pressure is applied to the tunnel at a unit pressure, $H + V$, the initial circumferential compressive stresses will be balanced by the tendency of the internal pressure to produce tension. Any additional internal pressure will produce apparent circumferential tension in the rock, and there will be a tendency for the rock particles to separate, and for existing natural seams, relieved of initial compression, to open.

The writer believes that (with a safety factor applied) this method of calculating the strength of pressure tunnels may be of some value in tunnel design and has prepared Fig. 8 to show the relation of depth of rock over-

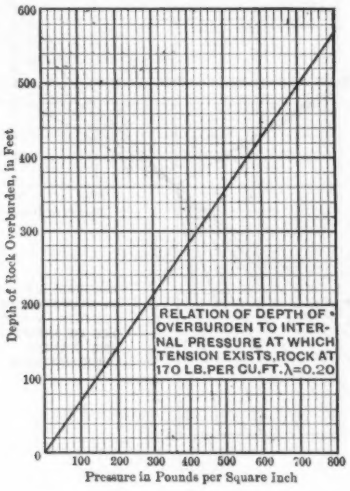


FIG. 8.

burden to the critical pressure at which apparent tension exists in the rock immediately surrounding the bore. It will be noted that no safety factor is included in the diagram. The factor of lateral deformation, λ, is taken as 0.20 and the weight of the rock at 170 lb. per cu. ft. Apparent tension exists whenever the head of water, in feet, is more than three and one-quarter times the head, or the rock overburden, in feet.

It must be borne in mind that any rock may contain seams, arched over, which will not be closed by the weight of the overburden. Such seams exposed to water pressure would allow a dangerous and possibly disastrous hydrostatic pressure to develop over large areas in the rock. For this reason, pressure tunnels ordinarily must be sealed with a lining as nearly watertight as practicable. Where the overburden is ample, the lining should be a dense concrete and the seams in the rock should be grouted under pressure. Where the tunnel approaches the surface, as in the case of a penstock, the lining should be of steel plate of whatever strength may be necessary to make up the deficiency in the strength of the rock, with a backing of porous concrete freely drained.

The writer believes that concrete placed between the steel lining and the rock in penstock tunnels should be a porous mix, with drains placed in it, in order to carry away any leakage through the lining and to relieve external ground-water pressure. (See Fig. 9.) If such construction is impracticable,

other provision must be made for protecting the steel against crushing by external pressure, such as angle or channel collars, or a ring of concrete inside the steel. Where porous concrete and drainage are practicable, no grout should be used in steel-lined penstock tunnels.

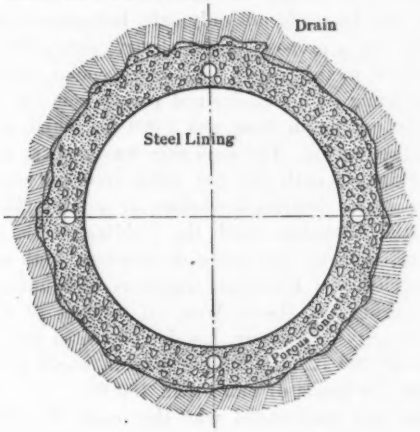


FIG. 9.

The concrete backing of a steel penstock tunnel lining need not have great strength, because (*a*) the shrinkage of the concrete on setting will be sufficient to render it ineffective, except to transmit direct stresses in radial directions (this makes the effective diameter of the tunnel the diameter of the excavation); and (*b*) ordinarily, the pressures to be transmitted in a radial direction are no greater than can be safely carried by a lean concrete.

DISCUSSION

B. F. JAKOBSEN,* M. AM. SOC. C. E. (by letter).—The author discusses an important question of design and one that needs to be studied in order that construction may be made not only safe, but economical. Several years ago, in connection with a proposed hydro-electric plant, the writer considered the problem discussed in this paper. An arrangement similar to Fig. 9 was contemplated, consisting of a well-drained porous concrete with a steel skin placed inside. The maximum head was 2 000 ft., which is equivalent to a pressure of 868 lb. per sq. in. The steel skin was to serve merely as a water-tight barrier, and its strength did not enter into the consideration. The principal question was to determine whether or not the skin was sufficiently elastic to permit the extension which the yielding of the concrete and the rock would produce, without developing dangerously high stress in it.

The recent experience of European engineers with unlined tunnels which were resorted to during the World War, on account of the high price of steel and the uncertainty of delivery, has demonstrated that such tunnels are not safe for heads of 300 ft., and most engineers would probably hesitate to use unlined tunnels for heads of more than 150 ft.

The writer does not understand why the usual formulas developed for thick cylinders do not fit this case. These formulas are derived on the assumption that the longitudinal stress is constant throughout the section, and when the external radius becomes infinite, the longitudinal deformation becomes zero, as the author assumes in Equation (4). Also, Equation (18) is the fundamental expression from which the stresses are derived by integration. For zero external pressure, Bach† gives the following formulas for the stresses x ft. from the center. Using the author's nomenclature:

$$L_t = \frac{m-2}{m} \frac{r_i^2}{r_a^2 - r_i^2} p \quad \ldots\ldots (26)$$

$$T_t = \frac{m-2}{m} \frac{r_i^2}{r_a^2 - r_i^2} p + \frac{m+1}{m} \frac{r_a^2 r_i^2}{r_a^2 - r_i^2} \frac{p}{x^2} \quad \ldots\ldots (27)$$

$$R_t = \frac{m-2}{m} \frac{r_i^2}{r_a^2 - r_i^2} p - \frac{m+1}{m} \frac{r_a^2 r_i^2}{r_a^2 - r_i^2} \frac{p}{x^2} \quad \ldots\ldots (28)$$

in which tension is positive and compression is negative, $\frac{1}{m}$ = Poisson's ratio, and p the water pressure. These are the true stresses, so that the true deformations are obtained by multiplying these stresses by E, the modulus of elasticity. If r_a, the outside or external radius, becomes infinite, $L_t = 0$, as it should be.

If, in Equation (27), $x = r_i$, T_t is the tangential stress at the inside of the tunnel, $T_t E$ is the elongation per unit length, and $T_t E r_i$ is the elongation of the radius. In the example of Fig. 6, $r_i = 6.5$, $r_a = \infty$, $p = 200$ lb. per sq. in., $m = 4$, and $E = 6\,000\,000$ lb. per sq. in., and the change of

* Cons. Engr., Fresno, Calif.

† "Elasticität und Festigkeit", 3d Edition, p. 492.

the radius is 0.003 in., whereas Equation (27) gives 0.00325 in. The stress in the steel skin would be $1.25\,p\,\frac{E_s}{E_c} = 1\,250$ lb. per sq. in., for $E_s = 30\,000\,000$ lb. per sq. in.

Even in first-class rock, the material nearest the tunnel is likely to have been deformed on account of the removal of the stress existing previously, and it seems reasonable to assume that this rock mass will deform or yield more to pressure than if the material had not been disturbed. It would seem safer, therefore, in order not to under-estimate the elongation to which the steel skin will be subjected, to assume arbitrarily a zone of no tangential stress nearest the tunnel. This zone should be assumed as extending a distance of, say, $3r_i$ from the center of the tunnel.

In this zone of no tangential stress, the pressure would vary inversely as the distance from the center of the tunnel, and it may be assumed that there is no lateral constraint, so that the rock will deform as if subject to compression in one direction and free to expand in the other directions.

On this assumption (see Fig. 10), the rock compression, x ft. from the center, is $\frac{p\,r_i}{x}$, and the deformation of the ring (of thickness $d\,x$) is

$$de = p\,\frac{r_i\,d\,x}{x\,E} \qquad (29)$$

Integrated between the limits, R and r_i, the deformation of the radius, r_i, is

$$e = \frac{p\,r_i}{E}\log_e(R - r_i) \qquad (30)$$

in which r_i is the radius of the steel lining.

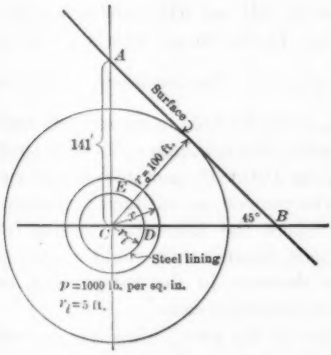


FIG. 10.

As an illustration, assume the condition shown in Fig. 10. Allowing 10 000 lb. per sq. in. in the steel lining, and computing by the ordinary cylinder formula, the required thickness is 6 in. If a steel skin as thin as can be handled economically is placed in fairly porous concrete, it is necessary to determine the stress in the steel lining due to the yielding of the rock, on the assumption that the steel lining does not carry any part of the stress.

In order to form an idea of the forces and stresses involved, it may be noted that the water pressure acting on the triangle, *ABC*, 1 ft. long, is 720 000 lb., on the assumption that the water is confined to the tunnel, giving an average shear intensity of 36.8 lb. per sq. in. along the plane, *BC*, provided no other forces are acting. The weight of the triangle, *ABC*, is nearly 1 700 000 lb., with rock weighing 170 lb. per cu. ft. The horizontal plane, *BC*, is not likely to be the plane in which the stresses are the highest, but the demonstration just given is sufficient to show that the arrangement may be considered, provided the rock is fairly sound. By assuming the stress

distribution to be the same as that in a thick cylinder, the elongation of the radius, r, from Equation (27), is found to be:

$$T_t \frac{r}{E} = \frac{r}{E}\left(\frac{1}{2}\frac{5^2 p}{100^2 - 5^2} + \frac{5}{4}\frac{100^2 p}{100^2 - 5^2}\right) = 0.001 \text{ ft.}$$

The tangential tension of the rock at the lining is practically 1 250 lb. per sq. in., and the tension in the lining, therefore, is 6 250 lb. per sq. in. If the external radius is assumed to be infinite, the deformation is decreased in the ratio, $\frac{9\ 975}{10\ 000}$, which is negligible.

The variation of T_t with x is shown in Table 1.

TABLE 1.

x, in feet.	$-Rt = Tt$, in pounds per square inch.	x, in feet.	$-Rt = Tt$, in pounds per square inch.	x, in feet.	$-Rt = Tt$, in pounds per square inch.
5	1 250	20	78.4	60	8.7
7.5	558	25	50.2	80	4.9
10	313	30	34.8	100	3.1
15	139	40	19.6	...	...

It is evident that, in this case, the rock will not withstand a tension of 1 250 lb. per sq. in. The pressure at point, D, due to the weight of the rock above D, is only $\frac{136 \times 170}{144} = 160$ lb. per sq. in. The tensile strength of good granite is about 600 lb. per sq. in., but it is hardly safe to assume any tensile strength, especially near the tunnel; therefore, the maximum allowable tension should not exceed the 160 lb. per sq. in. in Point D, and it would be much less in Point E. If a zone of no tangential tension, as outlined previously, is considered as extending to a distance where the tangential tension from Equation (27) is found to be within safe limits, this would neglect the tension which actually exists, due to a decrease in the compression, and, moreover, no such zone can be found in this particular case.

A value for the increase of the diameter of the steel lining may be found, as follows: Neglecting the tensile strength of the rock, the compression in the plane, BC, is roughly proportional to the height from any point to the surface. For points in the plane, AC, the compression is due to the lateral stress, or equals $\frac{V}{m}$, if V is the vertical pressure. As a safe average, 0.5 $\left(V + \frac{V}{m}\right)$, which, for $m = 4$, gives 0.625 V, may be taken. Then, for any value of x, in Fig. 10,

$$V_x = (141 - x)\frac{170}{144} \text{ lb. per sq. in.} \quad \ldots\ldots\ldots\ldots\ldots\ldots (31)$$

The total tangential tension over AB, in Fig. 11, where $AB = x - r$, or $x - 5$ in this case, is

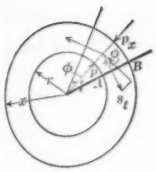


FIG. 11.

$$S_t = \frac{5}{8} \frac{(141 - x)\frac{170}{144} + (141 - 5)\frac{170}{144}}{2} (x - 5)$$

$$= \frac{5}{8} (163.5 - 0.59\,x)(x - 5) \ldots\ldots\ldots (32)$$

This tangential tension brings about a decrease in the radial pressures. Fig. 11 shows the relation between the three stresses, from which follows:

$$p\,r\,2\,\phi - p_x\,x\,2\,\phi = 2\,S_t \sin\phi$$

For ϕ approaching zero and using the value of S_t from Equation (32),

$$p_x = \frac{p\,r - \frac{5}{8}(163.5 - 0.59\,x)(x - 5)}{x} \ldots\ldots\ldots\ldots (33)$$

The relation between p_x and x is shown in Table 2.

TABLE 2.

x, in feet.	p_x, in pounds per square inch.	x, in feet.	p_x, in pounds per square inch.	x, in feet.	p_x, in pounds per square inch.
5	1 000	25	125.5	60	10.00
10	451	30	90.6	70	0.57
15	269	40	48 5	..	.....
20	174	50	24.6	..	.....

The stresses are plotted in Fig. 12.

For $x = 70$ ft., $p_x = 0$. The deformation, in this case, is

$$e = \frac{1}{E} \int_r^x p_x\,dx \ldots\ldots\ldots\ldots\ldots\ldots\ldots (34)$$

in which, $r = 5$ and $x = 70$. Integrating:

$$e = \frac{5}{8} \frac{1}{E} [p\,r \log_e (x - r) - 163.5\,(x - r) + 817.5 \log_e (x - r)$$

$$+ 0.295\,(x^2 - r^2) - 2.95\,(x - r)] = 0.00286 \text{ ft.}$$

The tension in the steel lining is, therefore, 17 900 lb. per sq. in. This, the writer believes, is a fair conservative value for the tension in the steel, under the circumstances, and the values obtained from Equation (27) are likely to be somewhat below the actual stresses. A design, such as that shown by Fig. 10, therefore, should be possible, even if the rock is only fairly good. With drain pipes, as shown by Fig. 9, and properly connected, there should be fair warning if leaks develop in the steel skin. Before using such a design as that shown by Fig. 10, the writer would determine roughly the deformation of the steel skin, on the assumption that the rock was fractured along the vertical plane, AC.

The author deserves credit for having submitted his ideas on pressure tunnels. Discussion of the subject cannot fail to be beneficial, and will lead

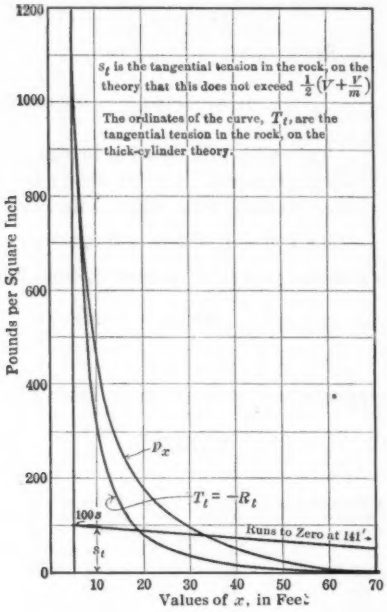


FIG. 12.

eventually to a more rational undertaking of the phenomena involved and, therefore, to more sensible designs, which would lower the cost without affecting adversely the safety of the tunnel.

JACOB FELD,* JUN. AM. SOC. C. E. (by letter).—The author is to be congratulated for presenting another application of the mathematical theory of stresses to civil engineering design. His demonstration of the subject is especially to be complimented because of the clear description of the assumptions made and of the limitations imposed by these assumptions. As the mathematical development of the subject may be simplified, the writer will show a shorter as well as a more rigid method of solving the problem, and also a more general solution, by the theory of least work.

Part I of the paper relates to a problem very similar to Lamé's solution of the apparent stresses in thick cylinders. The results, with changes in the end conditions, are the same. Some simpler methods for obtaining the author's results are as follows.

Considering the stresses acting on a unit volume (Fig. 2), the longitudinal deformation is equal to,

$$e_L = \frac{\lambda R_t}{E} + \frac{\lambda T_t}{E} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (35)$$

* Brooklyn, N. Y.

assuming that the material is isotropic, and, therefore, λ and E are constant throughout the mass. Assuming, with the author, that there is no longitudinal displacement, $e_L = 0$ gives the equation:

$$R_t = -T_t \quad \ldots\ldots (36)$$

Substituting in Equations (1) and (2), it is found that $R_a = -T_a$.

This result can be obtained directly, and more easily, from the assumption that the longitudinal stress is zero; for this is the case of simple shear, with zero stress along one axis, and, therefore, the stresses along the other two axes are equal in magnitude and opposite in direction. The case of simple shear is also a condition of constant volume; the work done is entirely in deformation of shape.

Considering an annulus, of radii, x and z, the internal pressure changes these radii to $x + \Delta x$ and $z + \Delta z$. The original volume is $\pi (z^2 - x^2)$; the final volume is $\pi [(z + \Delta z)^2 - (x + \Delta x)^2]$. The difference must be zero, or,

$$2 z \Delta z + (\Delta z)^2 - 2 x \Delta x - (\Delta x)^2 = 0 \quad \ldots\ldots (37)$$

But,

$$\Delta z = \frac{R_{az}}{E} z, \text{ and } \Delta x = \frac{R_{ar}}{E} x \quad \ldots\ldots (38)$$

and, disregarding the infinitesimals of the second order, Equation (37) becomes,

$$\frac{2 z^2}{E} R_{az} = \frac{2 x^2}{E} R_{ax}$$

or,

$$R_{ax} : R_{az} = z^2 : x^2 \quad \ldots\ldots (39)$$

When $z = r$, at the circumference of the tunnel, $R_{az} = R_{ar}$, and, therefore, $R_{ax} = \frac{r^2}{x^2} R_{ar}$, the author's Equation (23). A very similar method was used by Barlow in his investigation of the stress in thick cylinders.

Several points in the paper show that the conditions, $R_a = -T_a$ (the minus sign denotes that one stress is compression and the other is tension) and $R_{ax} = \frac{r^2}{x^2} R_{ar}$, satisfy the author's equations. Fig. 3 assumes that R_a and T_a are equal and opposite at the circumference of the tunnel, for the length of the arrow in Area B is equal and opposite to the first arrow in Area A. Equation (16) has no value, but is merely an identity. The constant, C, comes from Equation (13) which shows that $e = R_a C$, and, as e is the tangential unit deformation, or $\frac{T_t}{E}$, substituting for T_t from Equation (7), there results:

$$C = \frac{1}{E}\left[(1 - \lambda^2)\frac{T_a}{R_a} - (\lambda + \lambda^2)\right] \quad \ldots\ldots (40)$$

which shows that Equation (16) is an expression for $\frac{R_a}{T_a}$ in terms of itself. The author might have obtained the same result from his equations as follows:

Equations (2) and (12):

$$R_t = R_a - \lambda L_a - \lambda T_a = e E \ldots\ldots\ldots\ldots\ldots\ldots (41)$$

Equation (5):

$$L_a = \lambda T_a + \lambda R_a \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (42)$$

Equation (15):

$$eE = (1 - \lambda^2)\, T_a - (\lambda + \lambda^2)\, R_a \ldots\ldots\ldots\ldots\ldots (43)$$

Equating Equations (41) and (43):

$$R_a - \lambda\, L_a - \lambda\, T_a = (1 - \lambda^2)\, T_a - (\lambda + \lambda^2)\, R_a \ldots\ldots\ldots (44)$$

Substituting for L_a from Equation (42), the result is $R_a = T_a$

Equation (18) may now be solved easily:

$$R_{ax}\, d\, x - x\, d\, R_{ac} = - T_{ax}\, d\, x = - R_{ax}\, d\, x \ldots\ldots\ldots\ldots (45)$$

Simplifying,

$$2\, R_{ax}\, d\, x = x\, d\, R_{ax}$$

or,

$$\frac{d\, R_{ax}}{R_{ax}} = \frac{2\, dx}{x} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (46)$$

Integrating,

$$\log R_{ax} = 2 \log x + \log C = \log C\, x^2 \ldots\ldots\ldots\ldots\ldots (47)$$

and, therefore,

$$R_{ax} = C\, x^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (48)$$

To evaluate C, note that R_{ax} becomes R_{ar}, a known quantity, when x becomes r; therefore,

$$R_{ar} = C\, r^2$$

or,

$$C = \frac{R_{ar}}{r^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (49)$$

Substituting in Equation (48),

$$R_{ax} = \frac{x^2}{r^2}\ R_{ar},$$

which is Equation (23).

The curves may now be drawn for any chosen values of r and R_{ar}.*

A much easier method, which may also be solved with less detail for the more general case, is the application of the principle of least work. To state it in one of its forms: "For stable equilibrium, the potential energy of any system must be a minimum." The advantage of this method comes from the fact that the potential energy is equal to the work done by the external forces, and, therefore, the effect of the internal stresses may be disregarded.

To solve the first case, a pressure tunnel in a weightless rock (see Fig. 2), the only external force acting on the unit volume is R_{ax}. The deformation of the length, Δx, is $R_{ax} \frac{\Delta x}{E}$, and the work done in the annulus, of radius x, is:

$$\frac{1}{2}\, (R_{ax}) \left(R_{ax} \frac{\Delta x}{E} \right) (2 \pi x)$$

* A generalization of this method, including the effect of the exterior weight, is given in detail by Slocum and Hancock in "Strength of Materials", Chapter VII.

or,

$$\frac{\pi}{E}(R_{ax})^2 x\, d\, x$$

Integrating from r to infinity:

$$\text{Potential energy, } U = \int_r^\infty \frac{\pi}{E}(R_{ax})^2 x\, dx \ldots\ldots\ldots\ldots\ldots (50)$$

The principle of least work can be expressed by the requirement,

$$\frac{\delta U}{\delta R_{ax}} = 0.$$

Therefore,

$$\frac{\delta U}{\delta R_{ax}} = \frac{\delta}{\delta R_{ax}} \int_r^\infty \frac{\pi}{E}(R_{ax})^2 x\, dx \ldots\ldots\ldots\ldots\ldots (51)$$

$$\frac{2\pi}{E} \int_r^\infty (R_{ax})\, x\, dx = 0 \ldots\ldots\ldots\ldots\ldots (52)$$

The differentiation under the integral sign is allowable in this case, because $\int R_{ax}\, x\, dx$ is uniformly convergent for the range $R_{ar} \geq R_{ax} \geq 0$, and $\int (R_{ax})^2 x\, dx$ converges for one value of R_{ax}, for example, zero.*

To solve this equation, the form of the relation between R_{ax} and x must be assumed. The probable forms are the power and the exponential; the former gives the simpler solution. Let $R_{ax} = C x^n$:

$$\frac{2\pi}{E} \int_r^\infty x^{n+1} d\, x = 0 \ldots\ldots\ldots\ldots\ldots (53)$$

$$\int_r^\infty x^{n+1} d\, x = 0, \quad \left[\frac{x^{n+2}}{n+2}\right]_r^\infty = 0 \ldots\ldots\ldots\ldots\ldots (54)$$

$$\infty^{n+2} = r^{n+2} \ldots\ldots\ldots\ldots\ldots (55)$$

and as r is not infinite, the only possible value of n is -2. Therefore, $R_{ax} = \frac{C}{r^2}$; and, evaluating C as before,

$$R_{ax} = R_{ar} \frac{r^2}{x^2} \ldots\ldots\ldots\ldots\ldots (56)$$

The same result may be obtained directly from the expression for the potential energy, U, by letting $R_{ax} = Cx^n$.

$$U = \int_r^\infty \frac{\pi}{E}(R_{ax})^2 x\, d\, x; \quad R_{ax} = C x^n \ldots\ldots\ldots\ldots\ldots (57)$$

$$U = \int_r^\infty \frac{\pi}{E} C^2 x^{2n+1} d\, x = \frac{\pi C^2}{E} \int_r^\infty x^{2n+1} d\, x \ldots\ldots\ldots\ldots\ldots (58)$$

$$U = \frac{\pi C^2}{E(2n+2)} x^{2n+2} \Big]_r^\infty \ldots\ldots\ldots\ldots\ldots (59)$$

* See any advance work on the theory of the real variable, as Whittaker and Watson, "A Course of Modern Analysis"; Goursat, "Cours de Analyse", Vol. I; or, Pierpont, "Theory of the Function of a Real Variable."

$$\frac{\delta U}{\delta R_{ax}} = \frac{\delta U}{\delta x} \cdot \frac{d x}{d R_{ax}} = \frac{\delta U}{\delta x} \cdot \frac{1}{\frac{d R_{ax}}{d x}} = 0 \quad \ldots\ldots(60)$$

$$\frac{1}{\frac{d R_{ax}}{d x}} = \frac{1}{C n x^{n-1}}; \quad \frac{\delta U}{\delta x} = \frac{\pi C^2}{E (2 n + 2)} (x^{2n+1}) (2 n + 2) \Big]_r^{\infty} \quad \ldots(61)$$

$$\frac{\delta U}{\delta R_{ax}} = \frac{\pi C}{E n} \frac{x^{2n+1}}{x^{n-1}} \Big]_r^{\infty} = 0; \text{ or, } x^{n+2} \Big]_r^{\infty} = 0 \quad \ldots\ldots(62)$$

$$\infty^{n+2} - r^{n+2} = 0 \quad \ldots\ldots(63)$$

The only possible value for n is -2.

It can be shown that, if the weight of the rock is taken into account, the proper form for R_{ax} is $Cx^{-2} + B$. That this is true, without giving the mathematical details, may be seen from the fact that the expression for the work done in the mass now contains more than one term, but the only term involving R_{ax} is the same as in the preceding case. Therefore, the value of $\frac{d U}{d R_{ax}}$ is the same, and the value of n is -2. There are two constants because there are two end conditions.

To evaluate C and B, it is known that when $x = r$, R_{ax} (at the circumference of the tunnel) is $R_{ar} - w y \cos \theta$; and when $x = \frac{y}{\cos \theta}$ (at the surface), R_{ax} is zero. y is the vertical depth of the pressure tunnel below the top, or the depth of the overburden, having a density, w, and θ is the angle which the direction of r under consideration makes with the vertical.

Substituting the surface condition:

$$0 = \frac{C \cos^2 \theta}{y^2} + B, \text{ or } B = -\frac{C \cos^2 \theta}{y^2} \quad \ldots\ldots(64)$$

$$R_{ax} = C \left(\frac{1}{x^2} - \frac{\cos^2 \theta}{y^2} \right) \quad \ldots\ldots(65)$$

Substituting the other condition:

$$(R_{ar} - w y \cos \theta) = C \left(\frac{1}{r^2} - \frac{\cos^2 \theta}{y^2} \right) \quad \ldots\ldots(66)$$

$$C = \frac{r^2 y^2}{y^2 - r^2 \cos^2 \theta} (R_{ar} - w y \cos \theta) \quad \ldots\ldots(67)$$

$$R_{ax} = \frac{r^2}{x^2} (R_{ar} - w y \cos \theta) \frac{y^2 - x^2 \cos^2 \theta}{y^2 - r^2 \cos^2 \theta} \quad \ldots\ldots(68)$$

which is the general solution.

It is unfortunate that the author has not elaborated on the applications of his formulas, and it is to be hoped that he will do so in his closing discussion. It might have been much clearer to have noted that Equations (17) and (18) are derived from the usual thin-tube formulas, instead of using Fig. 4.

The value of any theoretical development depends entirely on how closely the assumptions approximate the actual conditions. A few facts about some recent discoveries concerning the stresses in rock may aid in evaluating the accuracy of the assumptions.

Professor Walter H. Bucher,* of the University of Cincinnati, has summarized the experimental work of L. Hartmann (1896) and the analytical work of O. Mohr (1900), covering the methods of failure of rock:

"(*a*) In all hard materials (except the most brittle ones), under tensional as well as compressional stresses, deformation by shearing takes place in two systems of intersecting planes of shearing.

"(*b*) Adjoining planes of one system are parallel.

"(*c*) The angle at which the two systems intersect is constant for any given material, that is, it is independent of the nature or intensity of the stresses involved.

"(*d*) For the same kind of material, this angle differs the more from 90° the harder and the more brittle the material is. * * *

"(*e*) If we consider tension as negative compression, the law governing the arrangement of the yield planes with reference to the principal axes of stress which will be referred to as Hartmann's Law, can be expressed as follows: In brittle materials, the acute angle formed by the shearing planes is bisected by the axis of maximum compression, and the obtuse angle by the axis of minimum compression which is generally negative, representing tension.

"(*f*) If the position of the principal axes changes from point to point, the shearing surfaces are warped. The less this is the case, *i. e.*, the more nearly homogeneous a material is, the more regular are the shearing planes.

"(*g*) The shearing planes do not originate simultaneously, and are not uniformly distributed."

In the case considered by the author, the maximum compression stress is along a vertical plane through the axis of the tunnel, for then (see Equation (68) θ is 180° and $\cos\theta = -1$. Failure under such conditions tends to occur along two shear planes, symmetrical about this plane, starting from the ends of the horizontal section of the tunnel. The vertical stress on the tunnel, therefore, is greater than that due to the weight of the overburden directly above it.

Mohr's theory of the ellipse of stress connects the angle of shearing with the ultimate tensile and compressive strengths by the relation:

$$\cos\theta = \frac{s_1 - s_2}{s_1 + s_2}.$$

Bucher states that by crushing small cubes in a vise, the angle of shear may be approximately determined experimentally, with remarkable accuracy.

Some of the characteristics of the angle of shear are that it is independent of the hardness of the material and also of the deformation of which the substance is capable below the elastic limit. If the tensile strength is greater than the crushing strength, the shearing angle is obtuse (for example, wood with the grain). The angle of shearing is more acute the more brittle the substance, and *vice versa*.

Karman's experiments,† determining stress-strain diagrams for marble and sandstone cylinders exposed to independent and simultaneous longitudinal and radial pressure, showed conclusively that the angle of shear increases

* *Journal of Geology*, Vol. 28 (1920), p. 712.

† *Zeitschrift* des Vereins deutscher Ingenieure, Vol. 55 (1911), pp. 1749–1757.

with an increase of radial pressure. That is, the radial pressure makes a material less brittle. This can easily be explained if it is realized that radial pressure permits a greater tensile stress in the outermost fibers.

The opposite case, cylindrical tubes exposed to internal radial pressure (the case considered by Mr. Dunn), was investigated by Mason in 1911,* in studying "Lüder's Lines in Mild Steel". Cylinders exposed to internal hydrostatic pressure and external hoop stress showed a decrease in shearing angle from 100° to 84°, as the hoop stress was increased.

The writer presents the foregoing facts in order to bring out the point that the strength of the rock overburden may not be its maximum tension value, but that the actual shear stress must be investigated. In other words, the author may be testing the wrong link in the chain in determining its true strength.

To illustrate the point further, the following is quoted from the paper entitled "Engineering Geology of the Catskill Water Supply," by Charles P. Berkey, Esq., and James F. Sanborn, M. Am. Soc. C. E.:†

"To prevent outward leakage from the [Rondout pressure] tunnel which is subject at this point to a head of about 700 ft., the concrete lining was made 3 ft. thick. This lining was placed inside a steel shell to protect the concrete during placing and setting. Dry packing placed outside the tunnel was grouted and the surrounding porous rock was impregnated with grout under high pressure."

Under a hydrostatic test, the shaft and tunnel being subjected to the working pressure, the concrete lining was ruptured for several hundred feet, between Shafts 3 and 4, where the overburden was about 350 ft. Fig. 8, indicates that a head of 700 ft. (roughly 300 lb. per sq. in.), requires an overburden of 220 ft. to prevent circumferential tension. The discrepancy between this and the actual value is too great to be explained by local weaknesses, especially as the failure was over such a great length.

William Cain,‡ M. Am. Soc. C. E. (by letter).—Equations (1) to (12) may be accepted, but objection can be urged to Equation (13) and the resulting conclusions. If e_r = unit deformation (or strain) in the direction of r from Equations (1) and (5):

$$e_r E = R_a - \lambda L_a - \lambda T_a = R_a - \lambda^2 (T_a + R_a) - \lambda T_a,$$

therefore,

$$e_r E = R_a (1 - \lambda^2) - (\lambda + \lambda^2) T_a.$$

Thus, e_r varies with both R_a and T_a, and cannot vary with R_a alone, as Equation (13) asserts, unless $T_a = R_a \times$ constant. Hence, this tacit assumption is made in writing Equation (13), and it is inevitable that Equation (16) should reach the same conclusions.

Reasoning from such an unwarranted premise, the author finally, by a tentative method, finds that $R_a = T_a$. This is so different from the well-known Lamé results, pertaining to a thick cylinder, as to challenge attention. Whether or not the ends of the cylinder walls carry any pressure, it can be

* *Proceedings*, Physical Soc. of London, Vol. 23 (1911), pp. 305–333.

† See p. 32.

‡ Prof. Emeritus of Math., Univ. of North Carolina, Chapel Hill, N. C.

assumed that plane sections perpendicular to the axis of the cylinder or tunnel, remain plane under pressure—at least for a central part, not too near the ends; and, further, that, for a long tunnel, there is no longitudinal deformation or strain; which assumption leads to Equation (5).

It seems to the writer that, for a long tunnel in homogeneous rock, the longitudinal unit stress, L_a, should be zero for the central part. In fact, for the solid rock endowed with weight, before the tunnel is driven, the weight is transmitted vertically, and there is no lateral thrust; and this remains true however small the weight per cubic unit is taken, and, for the weightless rock assumed in the first part of the paper, it seems reasonable that the longitudinal stress, L_a, is zero for any value of x. In the derivation of Lamé's formulas, it is only necessary to assume that L_a, for any transverse section, is constant, so that the assumption, $L_a = 0$, is only a particular case of the general solution.* For any constant value of L_a (zero included), the unit circumferential stress, T_{ax}, at the distance, x, from the center of the supposed cylinder, is given by the equation:

$$T_{ax} = \frac{r^2}{r_1^2 - r^2}\left(\frac{r_1^2}{x^2} + 1\right)p,$$

in which,

r = radius of internal surface of cylinder;
r_1 = radius of external surface of cylinder;
p = unit pressure on internal surface of cylinder.

Here, p and T_{ax} are both positive, although p is always compressive and T_{ax} always tensile. For the tunnel, the radius, r_1, will have to be taken as the depth of the rock, from the surface to the center of the tunnel, the part of the rock outside the cylinder, of radius r_1, being ignored. As the formula quoted assumes that the normal pressure on the outside of the supposed cylinder is zero, the result is approximate, although, when r_1 is large, it is practically true for the circumferential tension at the inside surface ($x = r$), which is the quantity particularly desired.

Designating this circumferential unit tension at the inside ($x = r$) by T_{ar}, and at the outside ($x = r_1$) by T_{ar_1}, the equation gives, for,

$$\frac{r_1}{r} = 10,\ T_{ar} = 1.02\,p\ ;\ T_{ar_1} = 0.02\,p\ ;$$

$$\frac{r_1}{r} = 20,\ T_{ar} = 1.005\,p\ ;\ T_{ar_1} = 0.005\,p\ ;$$

* * * *

$$\frac{r_1}{r} = \infty\ ,\ T_{ar} = p\ ;\ T_{ar_1} = 0.$$

The last result is evident from the formula; for the preceding value, write, for $x = r$,

$$T_{ar} = \frac{r_1^2 + r^2}{r_1^2 - r^2}p = \frac{1 + \left(\frac{r}{r_1}\right)^2}{1 - \left(\frac{r}{r_1}\right)^2}p\ ;$$

* Merriman's "Mechanics of Materials", Tenth Edition, Articles 149 and 150; Morley's "Strength of Materials", Art. 122.

from which it is seen that, as r_1 increases indefinitely, $\frac{r}{r_1}$ approaches zero indefinitely, and T_{ar} approaches p. Thus, when the depth, r_1, from the surface to the center of the tunnel, is equal to or greater than 10 r, the value of T_{ar} can be taken equal to p.

The curve given by the equation for T_{ax} is similar in shape to the curve, A, in Fig. 3; but it is not like the curve for T_a of Fig. 5 that has a decided elbow, which appears to be unjustified.

As it has been supposed that $L_a = 0$, the greatest strain (deformation) which is experienced in the direction of T_a occurs at the inside ($x = r$), and is,

$$e_t = \frac{T_{ar}}{E} + \frac{\lambda p}{E} = \frac{p}{E}\left(\frac{{r_1}^2 + r^2}{{r_1}^2 - r^2} + \lambda\right),$$

The value of Ee_t is slightly greater than the value of T_{ar}; but the difference for $\lambda = 0.2$ is small.

The value, Ee_t, is called by some authors the true stress. This is an unfortunate designation, as it is really only a measure of the true strain.

As Morley* points out:

"There are three theories as to when elastic failure takes place, namely,

"(1) For a certain value of the maximum principal stress.

"(2) For a certain value of the maximum principal strain.

"(3) For a certain value of the maximum shearing stress, this being proportional to the greatest difference between principal stresses."

The third theory seems to have the best of it, for ductile materials for combined stresses, such as torsion and flexure, tension and torsion, and compression and torsion, at least, to judge from the experiments thus far made.† For the problem of the thick cylinder under fluid pressure, Theories (1) and (2) practically agree, so that the first should suffice for a practical solution.

It is seen from what precedes that Mr. Dunn and the writer agree in the conclusion that the circumferential unit tension at the inside of the tunnel for $r_1 \geq 10\ r$, is equal to the unit water pressure, but differ as to the variation of this tension outside the tunnel; which variation, however, bears no practical significance as to the problem in hand.

The previous solution refers to a rock without weight. This weight must now be considered. Let Fig. 13 represent a vertical section at right angles to the axis of the tunnel. It can easily be shown that the weight of rock directly over the tunnel, after it has been excavated, can be carried by shear along vertical sides tangent to the tunnel. Thus, in Fig. 13, consider the rock contained between two vertical planes perpendicular to the axis of the tunnel and 1 ft. apart; its weight is, say, 2 r r_1 w, where w is the weight per

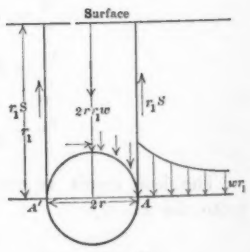


FIG. 13.

* "Strength of Materials", p. 33.

† In the *Journal*, West. Soc. of Engrs., November, 1912, there is an able paper, by Prof. O. H. Basquin, entitled, "The Circular Diagram of Stress and Its Application to the Theory of Internal Friction", in which the experiments on combined stresses are discussed from various standpoints.

cubic unit of the rock; and letting S = the unit shear of the rock, this weight can be sustained by the total shear on the two vertical faces (each of area $r_1 \times 1$) of $2\,r_1\,S$. Therefore, $2\,r_1\,S = 2\,r\,r_1\,w$ and $S = r\,w$.

If $r = 10$ ft. and $w = 170$ lb. per cu. ft., $S = 1\,700$ lb. per sq. ft. $= 142$ lb. per sq. in. This is only about one-tenth of the ultimate shearing resistance of stone, so that the shear on the two walls is amply able to carry the weight of all the rock directly over the tunnel.

This rock over the tunnel does not develop any lateral thrust, corresponding to grain pressure in a bin; but, because of the tendency to sink, it may develop arch action, and may give horizontal thrusts for supposed arches all the way down, as in the case of the grain bin.* However that may be, the part of the rock near and just over the tunnel—on account of this tendency to sink, particularly when r is large—will develop arch action; therefore, it may be confidently expected that there will be a small horizontal thrust on a small vertical section just above the crown of the tunnel, with corresponding small vertical forces to cause this thrust, as shown in Fig. 13. The magnitude of these forces cannot be given, neither can the magnitude of the vertical stresses on a horizontal diametral plane outside the tunnel; but, as regards the latter, at some distance from the tunnel such stresses should equal $w\,r_1$, because there the weight of rock is transmitted vertically. The stress from such a point should increase toward the tunnel, because of the weight transmitted by the shearing forces mentioned, by vertical planes over A and to its right, as shown roughly in Fig. 13. For the lower half of the tunnel, there is some transmitted stress from the sides, which diminishes as the bottom of the tunnel is approached, where it is probably zero.

If this reasoning is correct, there is not a circumferential thrust of the same magnitude around the inside of the tunnel, but a variable circumferential thrust; for at the bottom of the tunnel, it is probably zero; at the top, where it equals the horizontal thrust, it is small; but at $A\,A'$, it is very large, and certainly greater than $w\,r_1$.

From such considerations, the writer is unable to agree with the author that, when a tunnel is excavated, constant circumferential stresses next the tunnel are induced, or that the tangential stress at A and B, Fig. 7, equals $H + V$, with the subsequent conclusion.

According to the writer's view, the problem admits of only a roughly approximate solution, as follows:

(1) For $r_1 \geq 10\,r$, for a rock without weight enclosing the tunnel, in which the fluid pressure is p lb. per sq. ft., the circumferential tension induced at the inside of the tunnel is the same as the fluid pressure p lb. per sq. ft.

(2) When the rock is supposed to weigh w lb. per cu. ft., the horizontal thrusts at the top and bottom of the tunnel are so small that they can balance only a very small part of the circumferential tension, p; therefore, to be on the side of safety, it is well to regard such horizontal thrusts as zero. Then, at the top and bottom of the tunnel, the rock will be subjected to a

*See the writer's "Earth Pressure on Walls and Bins", pp. 208–217, for Pressures on Tunnel Linings in Earth, and pp. 218–221, for Grain Bins, where such action is considered.

tension of p lb. per sq. ft. If it can withstand this tension, no cracking will occur; otherwise, joints or seams will open.

At the sides of the tunnel, the tangential stress (compression) due to the weight of the rock is certainly greater than $w r_1$, and is probably much greater; so that if this stress is greater than p, there can be no tension there. If it is less than p, the difference will represent the tension that still exists.

Evidently, the top and bottom of the pressure tunnel are the critical points, and it is disturbing to think of being dependent on the tensile strength of the rock there for security; on that account, the writer will be delighted if some one can prove a more exact and comforting theory than he has outlined.

J. CHARLES RATHBUN,* M. AM. SOC. C. E. (by letter).—This paper advances the rather intricate problem of the analysis of stress in rock tunnels, in that it gives a general idea of the stress distribution in the lining due to internal pressure. However, there are limitations to the application of the formulas which it might be well to suggest.

The pressure tunnel on the Skagit River Power Project (the source of inspiration for the paper) passes through an extremely uniform rock formation. If this condition does not prevail, little value can be placed on the results of the author's analysis.

In the case of a tunnel lined with unreinforced concrete and with no steel inner lining, cracks due to tension in the concrete, not only from shrinkage, but also from internal water pressure, are very likely to develop. Considerable value can be placed on the author's formulas for estimating the tension due to pressure, and the danger of leakage from the cracks can be estimated.

If a tunnel is drilled through uniform material with overburden sufficiently heavy to prevent the top of the tunnel being forced up, it would appear that there is little change in the unit stress of steel linings of various thicknesses unless the lining is very thick. If this unit stress is high (beyond the elastic limit), is not the tunnel still safe? The steel cannot rupture.

It is evident that the material in a mass of solid rock is subjected to enormous stresses due to shrinkage when cooling from its molten state, as well as upheavals and other geological changes. It is practically impossible to compute or even estimate these stresses, but, in many cases, they are doubtless near the elastic limit of the material. The stress due to overburden can usually be estimated. The concrete lining as well as the steel lining is stressed due to shrinkage of the concrete on setting. If these stresses are less than the elastic limit of the material, the formulas derived by Mr. Dunn will apply. Even in that case, it must be kept in mind that only the change in stress in the rock and in the concrete and steel lining due to internal pressure, and not the final stress, has been computed.

The reference on page 1393 regarding the protection of the steel lining against crushing by external pressure might be interpreted to mean that the steel shell would fail by buckling under a pressure much less than that given

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by the ordinary formula for internal pressure in a pipe. As the concrete lining, even if on one side only, prevents lateral buckling of the steel, this simple formula can be applied.

CHARLES P. DUNN,* ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Feld's excellent mathematical discussion is appreciated; he has treated the problem more skilfully than the writer. The following statement by Mr. Feld merits particular attention:

"The writer presents the foregoing facts in order to bring out the point that the strength of the rock overburden may not be its maximum tension value, but that the actual shear stress must be investigated. In other words, the author may be testing the wrong link in the chain in determining its true strength."

Mr. Feld has not followed the writer exactly on this point. The theory developed in the paper is based on a conception of the rock possessing strength due to compressive stresses set up by its own weight, no allowance being made for any strength the rock might possess to resist tension or shear, the assumption being that, in practical design, rock must be considered as shattered and broken.

The writer, not being advised fully of the conditions, would not attempt to say what caused the failure of a part of the Rondout Pressure Tunnel, referred to by Mr. Feld, but he is quite positive that the failure could not have been caused by shear. Even if the shear is assumed to occur in vertical planes, the weight of rock included between two vertical planes more than balances the water pressure under the condition cited, 350-ft. overburden and a 700-ft. head of water.

Mr. Jakobsen's practical experience, and his thorough knowledge of the subject, make his discussion a welcome addition to the paper. On three tunnel designs, the writer has calculated the probable stress in steel linings and has found that, if the rock cover is sufficient to carry the load, the stresses in a thin steel lining, allowed by the elastic movement of the rock, are well within safe limits and that, assuming steel to be necessary for water-tightness, good design calls for as thin a lining as is practical. It is not practical to calculate exactly the stress in a steel lining, because, as Mr. Jakobsen has stated, the shattered condition in the zone of disturbed rock near the tunnel, and the shrinkage of the concrete lining in setting, may allow an expansion of the steel somewhat greater than theory would indicate.

It is difficult to specify the thinnest plate that is practicable to be used as a lining, and the thickness of the plate required depends on the skill of construction crews. The writer offers the following equation as fairly representing average practice: $t = 0.03\ D$, in which $t =$ the thickness, in inches, of the plate to be used for lining if no strength is required, the steel being used for a water seal only; and $D =$ the diameter of the tunnel, in feet.

The thickness of the concrete lining to be used must be determined for each individual case. A good rule for preliminary estimates, and a starting

* Designing Engr., Portland Ry., Light & Power Co., Portland, Ore.

point for design, is to make the mean thickness of the lining, in inches, equal the diameter of the tunnel, in feet.

The difference that Mr. Jakobsen points out between Bach's Equation (27) and the example given in Fig. 6 is due to error in scaling on the diagram, and is not an actual difference. A calculation of the example by the writer's Equation (22) gives 0.00325 in., which is the same as the result derived by using Bach's Equation (27).

Professor Cain rightly calls attention to the writer's crude mathematics, and suggests a more rigid reasoning. The writer, handicapped by having become "rusty" in the use of mathematics, was forced to solve the problem by experimenting with a great many arithmetical examples. Such methods, although laborious, have a peculiar value in that one has absolute confidence in his work when it is finished. It is gratifying to note that although Professor Cain properly disapproves of the writer's technique, the results obtained by him check with those in the paper.

The writer's equation, $R_a = T_a$, challenged by Professor Cain because it is different from the Lamé results, is checked as being correct later in his work where he develops the equation, $T_{ar} = p$. The internal pressure, p, is identical with R_{ar}, therefore, $R_{ar} = T_{ar}$, and as the writer has stated in the paper, any relation obtaining between, R_a and T_a, at the inner surface, must also obtain at the surface of any imaginary cylinder, x, within the material, in order that equilibrium of forces may exist.

The writer cannot agree with Professor Cain's statements that "for the solid rock, endowed with weight, before the tunnel is driven, the weight is transmitted vertically, and there is no lateral thrust", and, "this rock over the tunnel does not develop any lateral thrust, corresponding to grain pressure in a bin". This is equivalent to stating that the rock has zero factor of lateral deformation. It seems preferable to state that a lateral thrust exists, but no lateral deformation can occur, because the thrust is equal and opposite in all directions. The phenomenon of particles of rock popping or snapping off the walls of mine tunnels at great depths appears to be evidence that lateral thrust exists.

The writer does not concur in the idea that the compressive stress due to weight varies from a very small value at the top and bottom of the tunnel to a maximum at the sides. The entire theory of the paper is based on a conception of the rock as an elastic material which tends to deform laterally under stress. Throughout the development of the theory, the thought is kept in mind that elastic deformation must take place in such a manner that the work performed is a minimum, and internal and external forces are in equilibrium. That rock possesses elasticity and tends to deform laterally under stress is unquestioned by so many authorities that it may be accepted as an established fact. It seems inconceivable, therefore, that a pressure tunnel depends for its safety entirely on the ability of the rock near the crown to resist tension, as is indicated by Professor Cain. If the strength of pressure tunnels were measured in this manner, no pressure tunnels could be built, because

all natural rock in large masses is seamed and cracked, and for all practical purposes may be said to possess no tensile strength.

The writer believes that the pressure tunnel derives its strength, not from ability to resist tension, but from initial compression in the rock due to its own weight. Internal pressure, tending to produce tension in the rock, reduces this initial compression, and if the fluid pressure is sufficiently great to overbalance entirely the initial compression, actual tension appears, with resulting danger of failure.

Mr. Rathbun's brief discussion is evidently the result of much careful thought. The point brought out in regard to the buckling of steel lining under external ground-water pressure when the tunnel is empty may be discussed a little more completely with profit. The writer does not believe that the stresses in steel lining, due to external pressure, can be computed by the simple thin-cylinder formula, because the steel shell is restrained by the concrete surrounding it. It seems probable that by chance the external pressure may be applied over a restricted area, causing local buckling. This condition would make it illogical to apply the thin-cylinder formula, which is based on a uniform application of load throughout the entire circumference.

The writer thanks those who have given time and energy to the discussion of the paper, and is pleased that, although there are some differences of opinion, the discussions are substantially a check on the work.

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Paper No. 1528

A RIVER DIVERSION ON THE DELTA OF THE COLORADO IN RELATION TO IMPERIAL VALLEY, CALIFORNIA

By S. L. Rothery,* Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Paul M. Entenmann and S. L. Rothery.

Synopsis

This paper describes a diversion on the delta of the Colorado River in Baja California, Mexico. The reason for the diversion, the conditions that existed to affect its success, the considerations that determined the work undertaken, and the manner in which the diversion was completed, are discussed.

An initial stream flow that was only a small proportion of the flood discharge was used to start the diversion. The river made its own channel along the diverted route, eroding and transporting probably from 12 000 000 to 18 000 000 cu. yd. of soil, whereas, in the 4 miles of dredging operations, only 657 000 cu. yd. were excavated.

Historical

The courses of that part of the Colorado River in Mexico and of its delta effluents are wholly in its own alluvial silts which now replace what formerly was the northern end of the Gulf of California. From the present head of the Gulf, a great bed of alluvium extends north and includes the Salton Sea, in California. (See Fig. 1.) The Salton Sea is situated at what was formerly the northern end of the Gulf, and its bottom is about 275 ft. below mean sea level. The width of the alluvium bed is shown on Fig. 1, within the contour line marked "Ancient Beach Line."

In early ages, this turbid river deposited its silt burden in the narrow Gulf, building a deltaic ridge across it to the western shore and cutting off the northern end. The Colorado has since been distributing soil on this ridge, and the flow alternating down the slopes in cycles has produced depositions of stratified silt and alternate submergences and re-appearances of the valley

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bottom. When the flow was entirely down the Gulf slope, evaporation left the valley soils parched and arid.

Complete submergence to about 30 ft. above sea level, which is the elevation of the delta ridge at its western end, would have occurred again during the present century had not the agricultural possibilities of the valley led to the development, since 1900, of an extensive international irrigation project on which from 600 000 to 650 000 acres of this fertile alluvium are now irrigated. During 1905-06 the entire flow of the Colorado River was into the valley, and the present Salton Sea was then formed. This inflow, its cause, and the operations undertaken to force the river back on its delta ridge (at the location marked *A* on Fig. 1), have been described.*

Since that outbreak, the tendency of the river has been to flow down the valley slopes and to abandon its course to the Gulf. The definite changes in the course of the river shown at *B*, *C*, and *D*, on Fig. 1, occurred in 1909, 1918, and 1921, respectively, and are in that direction. To oppose this tendency, the levee system shown, has been built. Following the operations that returned the river to its course to the Gulf, in February, 1907, the levee of the California Development Company was built to deflect the flood waters farther to the south. In the following three to five years, the Ockerson, Saiz, and Volcano Lake Levees were built to the heights then deemed necessary. All these levees, having a total length of 65 miles, have had to be heightened and strengthened, from time to time, and continually repaired at the breaches made in them during floods.

The western 7 miles of the Volcano Lake Levee has been raised 10.5 ft. since 1908. During this period, the elevations of the flood waters have increased each year, until in June, 1921, they were 13.7 ft. higher than those of 13 years ago. Prior to 1920, more than $900 000 had been spent on this levee alone. Both the Volcano Lake Levee and the Ockerson Levee have been heavily revetted with rock over their entire river slopes, and some engineers advocate further raising and strengthening to keep ahead of the growing elevations of the deltaic ridge and the consequent higher flood levels.

Fig. 2 shows the elevating process of the river bed and its banks. The shaded portion is the blanket of silt deposited on the north bank of Bee River between September, 1918, and December, 1920, at locations several miles apart. During this period, only one flood of long duration occurred (June, 1920), that could have contributed most of this deposition. The flood water that overflows the river banks loses its velocity, due to spreading through the heavy growth of brush. The silt burden is deposited and the ground is thus elevated. Although sufficient channel capacity for a major part of the flow of the river is maintained, deposition tends to keep the river on the ridge, as each bank becomes, in effect, a thickened levee several hundred feet wide, well revetted with a dense growth of brush and trees through which the overflow can percolate but cannot erode channels in the slope. As the flood velocities diminish, the river bed is refilled with silt and sand-bars that deflect the flow

* "Irrigation and River Control in the Colorado River Delta", by H. T. Cory, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 1204.

of subsequent freshets, causing undercutting and caving of banks. These changes in the meandering of the river thus broaden as well as elevate the ridge.

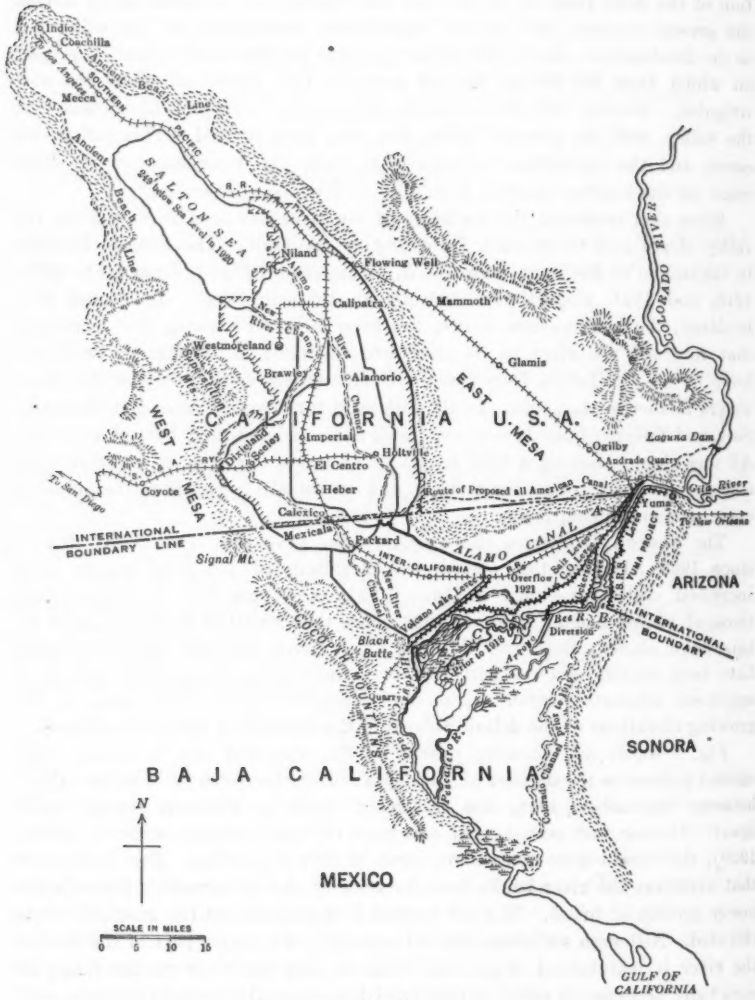


FIG. 1.

The large annual expenditures for levee building and maintenance are burdens, and the weakened condition of the levees during high floods causes apprehension and anxiety which are harmful both to land values and to the entrance of capital.

Diversion Possibilities

The elevations of the bed and banks of the Bee or Abejas River (the name given that part of the Colorado River between the abandoned channel of 1909 and Volcano Lake), are from 8 to 15 ft. higher along their present meander than they were on the meander of 1909-10, at about the same easterly or westerly locations. The banks are the highest elevations of the deltaic ridge, the ground sloping away from the river on each side.

Investigations during the winter of 1920-21 showed that it was feasible for the river to flow down the Gulf slope and that it might be turned from the ridge provided a sufficient quantity of water could be diverted to produce rapid erosion in the friable alluvium, so that the capacity of the initial channel along the diverted course would become enlarged to accommodate the increasing flood discharges of the river.

With complete diversion accomplished, there would be no flow to the Volcano Lake region, no flood waters against the Saiz and Volcano Lake Levees, and a lowering of the flood-water elevations along the Ockerson Levee should

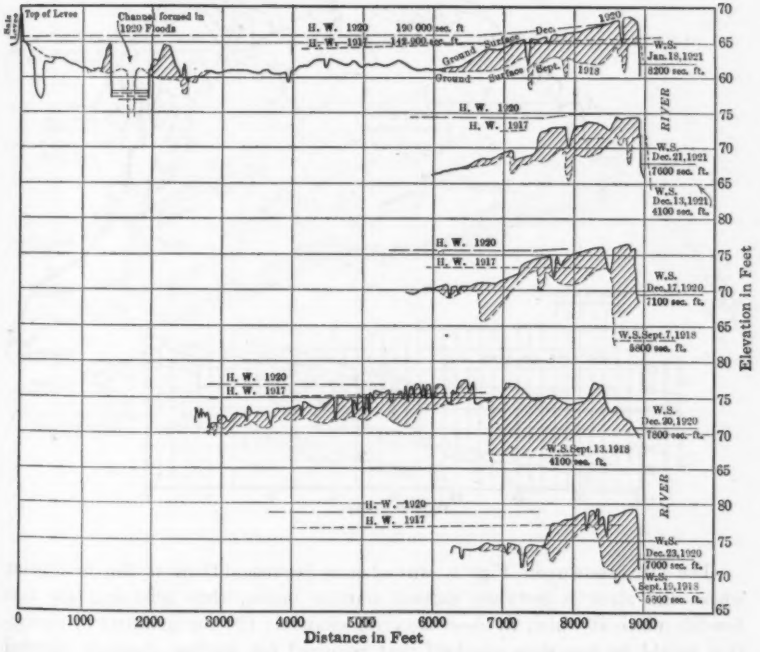


FIG. 2.

be expected. This betterment would be permanent for the Saiz and Volcano Lake Levees and of several years' duration for the Ockerson Levee, or until deposition along the meanders of the diverted course had again raised the

river to its present elevations up stream from the diversion point. Before such a period passes, however, it is hoped that permanent relief from floods will have been attained by the construction of the proposed Boulder Canyon Dam, more than 300 miles up stream from the delta.

The slope of the river, from the International Boundary Line to the western end of the Saiz Levee, near Volcano Lake, was taken by noting at 11 A. M., on December 21, 1920, the water surface elevations at these two extreme points and at four intermediate points. The slopes varied between 0.85 ft. and 1.25 ft. per mile.

To determine the site for the diversion, surveys of both banks of the Bee River westerly from the Ockerson Levee were made. Profiles were also run across the south bank toward the old 1909 channel, which is visible in places, and toward the upper ends of deltaic effluents, about 7 miles westerly.

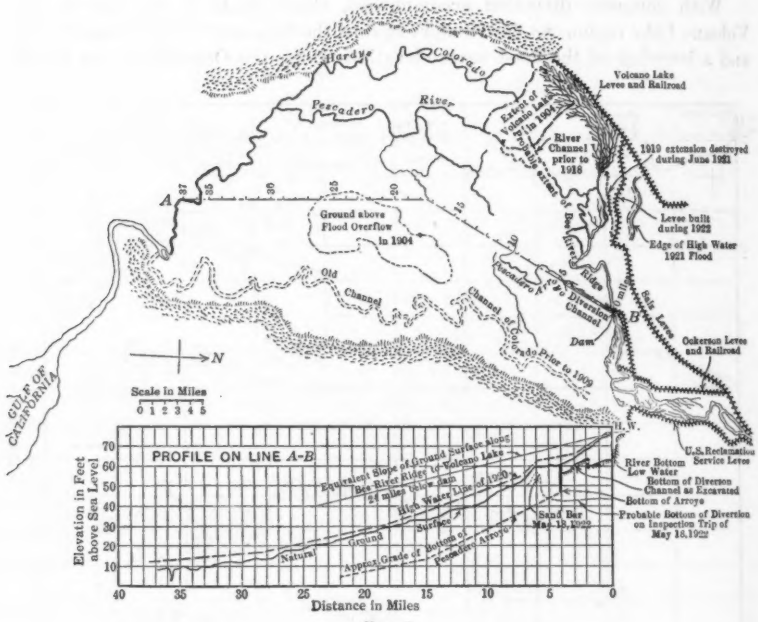


FIG. 3.

The site, shown on Fig. 1, was chosen because (1) it is the first point where the river is between defined narrow banks, thus affording the best feasible place at which to close the river channel; (2) the quantity of excavation would be less than one-half that required for similar channels directed toward the old 1909 channel; and (3) the meandering of the river, to the south and southeast of the Ockerson Levee, was so wide and changeable that the inlet to excavated channels might not remain connected with the main river.

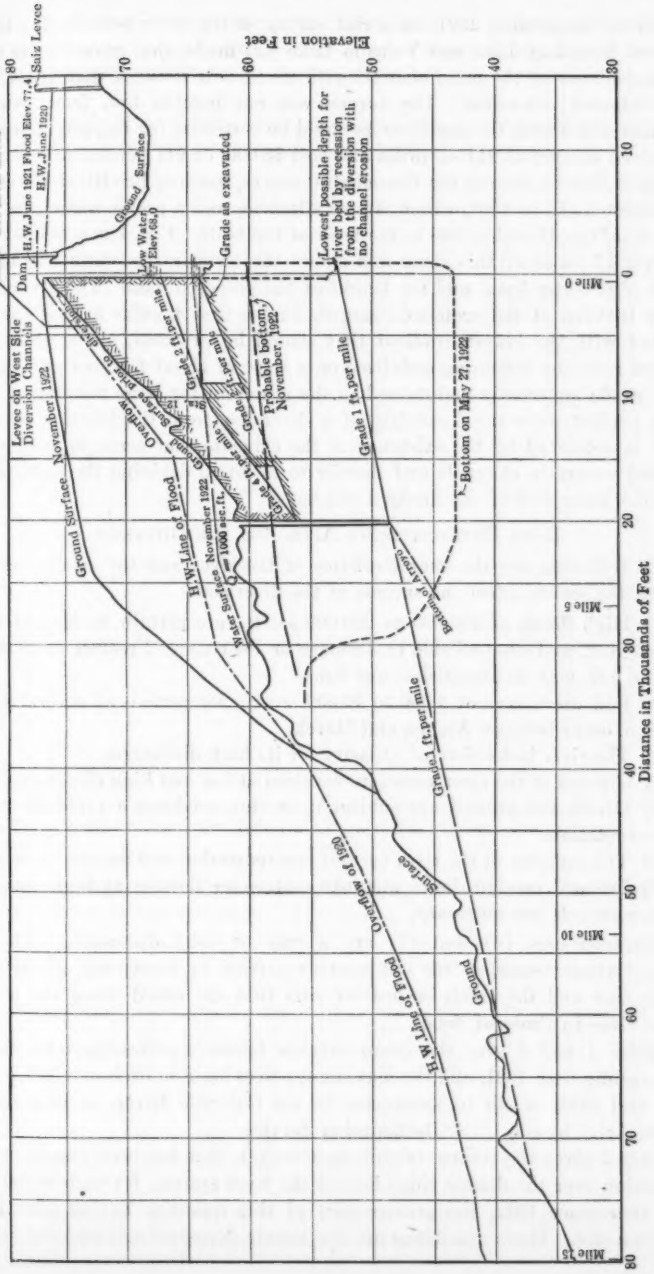

Elevation in Feet
Distance in Thousands of Feet
Bee River Levee
Dam
Levee on West Side Diversion Channels
Ground Surface
Ground Surface November 1922
Overflow Spillway
Grade 2 ft. per mile
Grade 1 ft. per mile
Probable bottom, November, 1922
Grade as excavated
Lowest possible depth for river bed by recession from the diversion with no channel erosion
Bottom on May 27, 1922
Bottom of Canyon
Grade 1 ft. per mile
Ground Surface
Original Flow of 1920
H.W. Line of Flood
Water Surface November 1922 1000 sec. ft.
H.W. June 1921 Flood Elev. 77.4 ft.
H.W. June 1920
Low Water Elev. (60.)
Mile 0
Mile 5
Mile 10
Mile 15

FIG. 4.

During September, 1921, an aerial survey of the river between the International Boundary Line and Volcano Lake was made, that gave a more comprehensive view of the main channel with its effluent streams, than could have been obtained otherwise. The survey was not precise, but, from existing traverses and levees, the small errors could be corrected for mapping purposes. It covered an area of 214 sq. miles and cost $3 000, or $14 per sq. mile.

Fig. 3 shows a map of the Pescadero Country, south of Bee River, the extent of Volcano Lake in 1904, which is now silted in, and a profile across the Gulf slope from the diversion site to the head of the Gulf. Fig. 4 is a profile along the first 15 miles of this slope and shows the comparative elevations of the levees, high-water lines, and the height of the proposed dam. The grade lines of the bottoms of the proposed channels in the first 4 miles are also shown, together with the lowest gradient that should be expected, owing to erosion and bed recession becoming stabilized on a slope of about 1 ft. per mile, which is the grade apparently maintained in the alluvium for large volumes of flow. These profiles show the feasibility of a diversion from the location selected, which is enhanced by the existence of the effluents that serve to confine the diverted waters in channels and thereby to assist in draining the water away from the lower end of the dredged channels.

River Characteristics Affecting the Diversion

The following are the characteristics of the river and the conditions that exist at the site to affect the success of the diversion:

(1) High floods of 100 000 to 200 000 sec-ft., are certain in May or June of each year, and are probable in January or February. Freshets of 10 000 to 60 000 sec-ft. may be expected at any time.

(2) Low discharges of 4 000 to 10 000 sec-ft., for periods of several weeks' duration, occur between August and March.

(3) The river lacks channel capacity for its high discharges.

(4) The bed of the river varies in elevation at low and high discharges.

(5) Overflowed ground not subject to erosion produces a rapid growth of dense vegetation.

(6) The capacity of the river for soil transportation and deposition is great.

(7) For soil erosion, large quantities of water flowing at high velocities within channels are necessary.

Characteristics (6) and (7) are worthy of brief discussion. The soil transportation comprises the silt content carried in suspension at all times of the year and the sands or heavier silts that are rolled along the bottom of the river in times of flood.

Tables 1 and 2 give the mean average monthly percentages for the 10 years ending with 1921, and the maximum percentages in each month for 1919, 1920, and 1921, of silt in suspension in the Colorado River, as measured at Yuma, Ariz., by the U. S. Reclamation Service.

Table 3 gives the volume of soil, in acre-feet, that has been transported in suspension over the deltaic ridge behind the levee system, for each month and each year since 1912, the greater part of this quantity having passed the diversion site. These quantities are for loosely deposited soil and were calcu-

lated by using the formula adopted by the office of U. S. Reclamation Service at Yuma, which is: Acre-feet of silt = acre-feet of discharge × percentage of silt by weight × 0.727.

TABLE 1.—Monthly Mean Percentages of Silt (by Weight), in Colorado River Discharges for 1912 to 1921, Inclusive.

Month.	1912.	1913.	1914.	1915.	1916.	1917.	1918.	1919.	1920.	1921.
January	0.28	0.23	0.48	0.63	1.77	0.48	0.15	0.15	0.60	0.22
February	0.47	0.35	0.78	1.26	1.59	0.31	0.15	0.32	1.35	0.18
March	0.88	0.62	0.95	0.97	1.62	0.49	0.96	0.58	1.23	0.52
April	1.17	1.26	1.23	1.25	1.20	1.03	0.39	0.93	1.02	0.34
May	1.15	1.17	0.98	1.06	0.93	0.98	0.54	0.50	0.94	0.80
June	0.50	1.06	0.48	1.04	0.66	0.54	0.53	0.54	0.66	0.50
July	1.07	1.17	1.00	1.24	0.65	0.46	1.01	1.67	0.66	0.67
August	1.22	0.65	1.20	0.74	1.95	0.68	0.68	2.24	0.92	2.12
September	0.67	0.60	0.68	0.21	0.87	0.30	0.63	1.03	0.60	1.17
October	0.74	0.70	0.88	0.45	1.91	0.29	0.48	0.72	0.26	0.43
November	0.79	0.52	0.70	0.35	0.68	0.14	0.34	0.54	0.39	0.32
December	0.39	0.38	0.86	0.33	0.32	0.13	0.28	1.22	0.20	0.55

TABLE 2.—Monthly Maximum Percentages for 1919 to 1921, Inclusive.

Month.	1919.	1920.	1921.
January	0.37	1.25	0.59
February	0.94	3.32	0.37
March	2.40	2.44	0.94
April	1.58	1.74	0.55
May	0.84	1.48	1.08
June	0.91	1.13	0.89
July	4.82	2.02	1.20
August	3.74	1.78	3.62
September	2.17	.....	4.01
October	1.97	0.43	0.91
November	0.87	.....	0.37
December	2.35	0.40	1.62

The coefficient, 0.727, is the ratio of weight of 1 cu. ft. of water to that of 1 cu. ft. of loosely deposited silt $= \frac{62.4}{86} = 0.727$.

TABLE 3.—Monthly Volumes of Soil, in Acre-Feet, Transported in Suspension to the Colorado River Delta, for 1912 to 1921, Inclusive.

Month.	1912.	1913.	1914.	1915.	1916.	1917.	1918.	1919.	1920.	1921.
January	477	225	1 298	2 119	32 770	1 581	294	114	2 419	446
February	1 136	587	3 167	13 040	17 184	712	175	633	20 057	319
March	4 438	1 841	5 355	5 599	24 121	1 578	5 572	1 493	8 146	2 396
April	9 618	12 311	10 613	14 637	16 451	10 217	1 399	6 604	7 081	1 531
May	19 875	18 819	21 952	21 295	22 247	19 932	5 835	7 091	17 441	14 205
June	22 765	20 439	21 994	19 802	15 602	20 000	13 075	6 910	35 559	23 095
July	21 117	9 395	21 118	15 147	9 813	18 436	17 565	11 144	11 084	12 350
August	10 910	1 859	10 018	2 650	20 465	6 423	2 098	5 167	5 089	29 609
September	2 124	1 579	1 961	112	3 345	625	479	368	1 030	11 451
October	2 791	2 530	4 398	905	20 318	512	821	624	306	1 204
November	3 424	1 493	2 600	615	2 920	286	848	615	137	679
December	872	894	4 521	584	1 028	282	664	7 084	389	1 871
Yearly totals	99 547	71 972	108 995	96 505	186 259	80 584	48 825	47 847	108 738	99 156

These figures total 948 428 acre-ft. for the 10-year period. Had 70% of this quantity been deposited on the Bee River Ridge behind the levee system, over a meandering width of 4 miles and a length of 25 miles, this area would have been raised more than 10 ft. since 1912.

The transportation of bed sand is not included in the quantities, as the percentages of silt given are a measure of the turbidity. The depth of scour of the river at Yuma and at Andrade, Calif., during periods of high flood, is as much as 20 ft. lower than the elevation of the normal river bed during low-water periods. Assuming a conservative deepening of 9 ft. for a width of only 600 ft., the volume of soil moved in each 100 miles of the river is 105 600 000 cu. yd. As much of this soil probably falls behind the traveling flood wave to fill the river bed again at locations down stream from its former position, its travel toward its destination on the delta may not be as rapid as that of soil transported in suspension, but undoubtedly great quantities are moved along the river in this manner.

The capacity of the river for erosion in its own alluvium is best conceived by an example. Table 2 shows the silt content to have been more than 4% and, for most of the year, less than 0.5 per cent. If a small part of the flow is diverted through its own alluvial deposit and its velocity is greatly increased, this part has the power to absorb and transport in suspension a higher proportion of silt. Assume that it gathers only 1% additional silt and that the flow is only 2 700 sec-ft. This flow is 100 cu. yd. of water per sec., which is equivalent to 1 cu. yd. of soil transported per second, or 86 400 cu. yd. per day. With such rapid erosion, provided increasing volumes of water are available to the diverted course, any channel can be quickly enlarged so that the entire river flow will soon follow the new course, especially if its old direction on flatter gradients is effectively blocked.

The capacity of the river for soil deposition, or the forming of silt bars, is conversely illustrated. If the flow was into ponded water, or was stilled by dense foliage of thick vegetation, 86 400 cu. yd. of soil would be deposited daily for each 2 700 sec-ft. of inflow charged with a turbidity of only 1 per cent. Confine any such possibility for deposition in a narrow channel, and it will quickly be silted in and become non-existent.

Work Undertaken to Commence the Diversion

The work to be done was based on the following conclusions:

1.—An initial flow of 5 000 to 10 000 sec-ft. of diverted water, having a depth of several feet, and confined to a channel having a slope that would produce erosional velocities in the alluvial soils, was judged to be the minimum necessary to commence erosion and to maintain flow conditions. A shallow depth of flow would be of no service; therefore, the bottom of an excavated channel must be well below the water surface of the river. The water surface of the river was about Elevation 68 for low-water conditions and Elevation 77.4 during the high flood of June, 1921. (See Fig. 4.) The bed of the river was at Elevation 62 for low water and was probably scoured to about Elevation 40 or Elevation 45 during high floods.

2.—The minimum excavation necessary to produce this flow required that the channel have maximum hydraulic radii for depths of 4 to 8 ft. of water. Such a channel would be semi-circular in section, but a rectangular section of the same top width and depth was favored, as undercutting was desired.

3.—As the diverted initial flow is only one-fortieth to one-twentieth of that of high flood, it is apparent that, to make the diversion complete, twenty to thirty times the yardage excavated, must be eroded and transported by the river in following the diverted route.

4.—An excavated width of more than 200 ft. would be necessary for the required initial flow. The dumping of the excavated soil limited the width to about 100 ft.; comparisons of discharges and velocities for one channel, 216 ft. wide, for two parallel channels, each 108 ft. wide, and for three parallel channels, each 72 ft. wide, were made, as shown in Table 4, for each foot-depth of flow.

TABLE 4.*

Width by depth, in feet.	Wet areas, in square feet.	Wet perimeters, in feet.	Hydraulic radii.	Maximum silting velocities, in feet per second.	Velocities for 52.1 ft. to mile, in feet per second.	Discharge, in second-feet.
For Equivalent Single Channel, 216 Ft. Wide.						
216 by 1	216	218	0.98	0.98	1.23	266
216 by 2	432	220	1.96	1.53	2.02	873
216 by 3	648	222	2.92	1.98	2.68	1 737
216 by 4	864	224	3.85	2.38	3.25	2 808
216 by 5	1 080	226	4.78	2.74	3.76	4 060
216 by 6	1 296	228	5.70	3.08	4.23	5 482
216 by 7	1 512	230	6.57	3.40	4.64	7 015
216 by 8	1 728	232	7.45	3.71	5.02	8 675
For Each of Two Channels, 108 Ft. Wide.						
108 by 1	108	110	0.98	0.98	1.20	259
108 by 2	216	112	1.93	1.53	2.00	864
108 by 3	324	114	2.84	1.98	2.64	1 711
108 by 4	432	116	3.72	2.38	3.18	2 748
108 by 5	540	118	4.57	2.74	3.65	3 942
108 by 6	648	120	5.40	3.08	4.08	5 288
108 by 7	756	122	6.20	3.40	4.47	6 759
108 by 8	864	124	6.97	3.71	4.82	8 329
For Each of Three Channels, 72 Ft. Wide.						
72 by 1	72	74	0.97	0.98	1.20	259
72 by 2	144	76	1.89	1.53	1.98	855
72 by 3	216	78	2.77	1.98	2.59	1 678
72 by 4	288	80	3.60	2.38	3.11	2 687
72 by 5	360	82	4.39	2.74	3.56	3 845
72 by 6	432	84	5.14	3.08	3.95	5 119
72 by 7	504	86	5.86	3.40	4.31	6 517
72 by 8	576	88	6.55	3.71	4.61	7 966

* Value of n used is 0.0225.

It was decided to excavate the two 108-ft. channels, in preference to the 72-ft. widths, as there would be less likelihood of the larger channels becoming choked with the great masses of floating driftwood. When dredging was com-

menced on the first channel, the bottom width was reduced to 93 ft., with side slopes of $\frac{1}{4}$: 1, in order that the dumping distance in the deepest excavation would not be too great for the 50-ft. boom on the dredges.

5.—As shown in Fig. 4, the original plan contemplated a grade of 2 ft. per mile, leaving the river at Elevation 67, 10 ft. below the banks, and changing to a grade of 4 ft. per mile at Station 112. After excavation had progressed up stream from the arroyo toward Station 112, it was decided to lower the grade 3 ft. to Elevation 64, at the river. This explains the level grade line shown.

To eliminate this level grade line a further improvement was begun. As the first channel was nearing completion in December, 1921, a cut, 60 ft. wide, was made on the grade of 2 ft. per mile from Elevation 64, thus effectively lowering the original bottom 3 ft. As shown on Fig. 4, this cut was never completed, however, and as it developed later, no change from the original grade of 2 ft. per mile from Elevation 67 was necessary, as there would have been an entry head of 8 ft. when the cut was opened.

It was planned to begin the diversion early in January, when low-water conditions usually prevail and when the river could be controlled. At that time of the year, the flow is sometimes as small as 5 000 sec-ft. and, for many months, it may continue at less than 20 000 sec-ft. Under these conditions, a dam could be built below the inlet of the diversion, and such discharges as were available could be used to begin recession and enlargement of the channel prior to the annual summer flood that has scouring velocities averaging 8 ft. per sec., on its present slope of about 1 ft. per mile.

As the initial capacity of the excavated channel was only a small proportion of the capacity of the river channel during flood, and as the river was on a ridge, no dam could impound the flood flow. Discharge in excess of the capacity of the diversion channel would quickly overflow the river banks and, for this reason, the dam was built only 3 to 4 ft. higher than the banks, and a small levee, of similar height, was built along the north bank from the dam to the Ockerson Levee, in order to confine the overflow to the south bank, up stream from the diversion channel.

The foundations and end abutments of the dam could not be made impervious because of the nature of the soil. A strong obstruction in the river channel was also not considered desirable until sufficient enlargement had been demonstrated because, if the levee along the north bank should fail at any point, the river might change its direction to the north and abandon its course into the diversion channels. For high freshets prior to the annual flood, this danger did not exist, as such freshets only lasted a few days, and the topography and dense growth to the north are such that a large stream flow with a falling gauge could not cause erosion.

Because of this element of danger, the dam was made the weak link, its function being to divert the low-river flow so that as much erosion as possible could be obtained by freshets and by the increasing flow of the annual flood. If these quantities happened to increase gradually, the complete diversion might have been made without the breaking of the dam. It would seem, also,

that for such ideal conditions there must be a free outlet at the south end of the channels, and the fall of the country makes this appear to be feasible.

Fig. 5 shows the work that was done. Three channels are shown, but only the east channel was made sufficiently deep and long to function effectively prior to the June flood. The other two channels were not intended to be opened to the river until prolonged high water. Fig. 6 is a view of the inlets of the three channels, Channel No. 3 being in the foreground, and Channel No. 1 and Bee River in the background.

The excavation was made with three 1.5-cu. yd. draglines and one 1-cu. yd. dragline. Before the end of January, 657 093 cu. yd. had been excavated by working continuously day and night. Of this excavation, 451 637 cu. yd. was from the main or east channel.

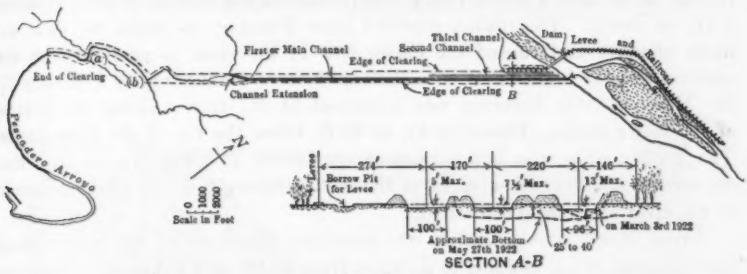


FIG. 5.

Contracts were made with Japanese laborers for clearing the ground along the route of the diversion and for 3½ miles along the arroyo beyond the lower end of the excavated channel. In all, 583.44 acres were cleared south of the river. A cleared strip, comprising 74.52 acres, was made along the north bank of Bee River for the levee and for the railroad extension from the Ockerson Levee. The railroad track was first laid on the surface of the ground, in order to deliver materials and equipment for the dam as quickly as possible. Pile-driving for the trestle of the dam was commenced on December 1, 1921, and by December 24, the trestle had been completed and the track laid across the river.

THE DIVERSION IN PROGRESS

It was expected that the channel would be opened to the river, and that the dam would be completed during January, 1922, at which time low flow conditions usually occur. However, a flash flood on December 25, 1921, destroyed the trestle which was then nearing completion. This flood was followed by another on January 4 or 5, 1922, the water surface in the river reaching Elevation 75.55, less than 2 ft. below the top of the banks. Advantage was taken of this high water, and the barrier, left in the first channel 3 700 ft. from the river, was cut at 7:20 P. M., on January 5. On the morning of January 6, the channel was discharging 7 000 to 8 000 sec-ft. at a velocity of about 6 ft. per sec. Fig. 7 is a view of the diversion channel

before the barrier was broken on January 5, 1922. Fig. 8 is a view of the channel, down stream from the inlet on January 10, 1922. The freshet had passed by January 8, and the profile (Fig. 9), taken on January 20, shows how much the bottom had been lowered in the first day or two.

From January to May, the discharges in the diversion channel were metered daily at a point 3 700 ft. from Bee River, and gauges, set along the channel and in the river, were also read. These observations shown on Fig. 10, revealed the progress of the diversion.

From January 15 to February 11, 1922, the discharge of the river was very low, and only a small quantity of water passed through the diversion. A second freshet with twin peaks, shown on Fig. 10, reached a maximum of 14 800 sec-ft. on February 14, and lasted 10 days. This freshet made the repairs to the dam a difficult task, but it lowered the bottom of the cut about 4 ft., as shown. The freshet subsided after February 20, when the dam was made effective and caused the entire flow of the river to pass through the channel and the lowering of an additional 6 ft. of the bottom, by February 27. By March 7, this lowering was increased 3½ ft., thus making the bottom at the meter station, Elevation 47, or 30 ft. below the top of the river banks. The profile along the bottom, taken on March 14 (Fig. 9), showed that the recession which was evident at the meter station, had extended up stream to the river and also down stream.

From March 6 to 18, 1922, the lessening discharge of the river caused the discharge in the channel to decrease from 11 400 to 5 500 sec-ft. However, the bottom did not silt in, but remained about at Elevation 47, and the pondage up stream from the dam drained into the cut, leaving the dam high and dry.

A third freshet occurred on March 20, its arrival being very sudden. By 5 A. M., a flood wave of 21 000 sec-ft. had arrived at the diversion, the discharge rising from 6 000 to 21 000 sec-ft., without any warning from up-stream sources. It was thought that the dam would withhold the strain on it, but on account of serious sloughing, a small gap in the middle of the dam was made with explosives. However, before the gap became effective, the north end of the dam failed at 7:00 A. M., and, in a few minutes, 130 ft. of it had washed away, with the river cutting around it into the north bank. The failure of the dam, together with the rapid fall of the flood wave, was responsible for the small discharges diverted and for the raising of the bottom about 5 ft. before March 23.

Fortunately, for the work on the cut, although not for the cost of repairing the dam, the freshet increased, and on March 26, the discharge had reached a peak of 30 800 sec-ft. and remained in excess of 15 000 sec-ft. until April 6. The significance of this third freshet was the evident increase in that proportion of the discharge of the river that followed the diverted course of the channel when no obstruction was offered by the dam. This forecasted the success of the diversion.

The break in the dam was closed by April 11, and the discharge of the river decreased from 16 000 sec-ft. on April 12, to less than 9 000 sec-ft., on April 28. The first water of the spring flood arrived on April 30, when the



FIG. 6.—VIEW OF INLETS OF THE THREE CHANNELS.

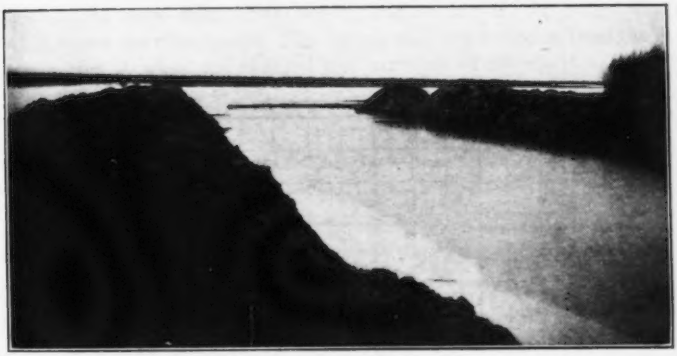


FIG. 7.—DIVERSION CHANNEL BEFORE BARRIER WAS BROKEN, JANUARY 5, 1922. SAND BAG DAM ACROSS INLET.

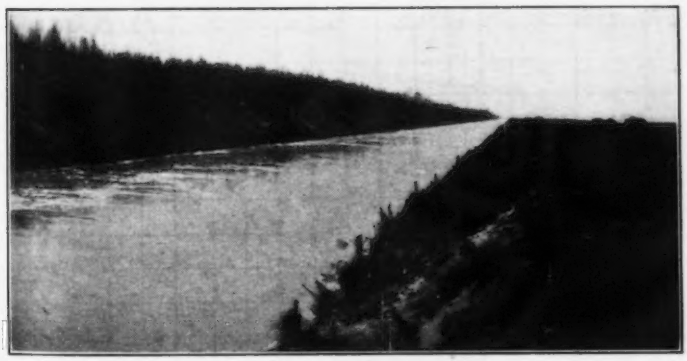


FIG. 8.—DIVERSION CHANNEL DOWN STREAM FROM INLET, JANUARY 10, 1922.

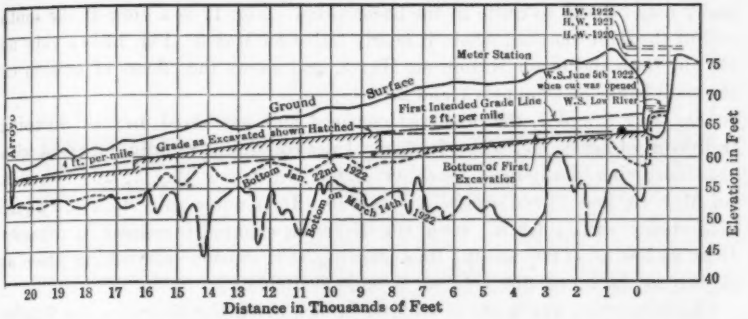


FIG. 9.

gauge showed a sudden rise of 3.2 ft. to Elevation 75.4. On May 1, the cut was discharging more than 27 000 sec-ft., and on May 2, the gauge was 76.1 ft., or 1 ft. below the river banks. Fig. 11 is a view down stream from the meter station on May 3, when the channel was carrying 29 560 sec-ft. Channels 1

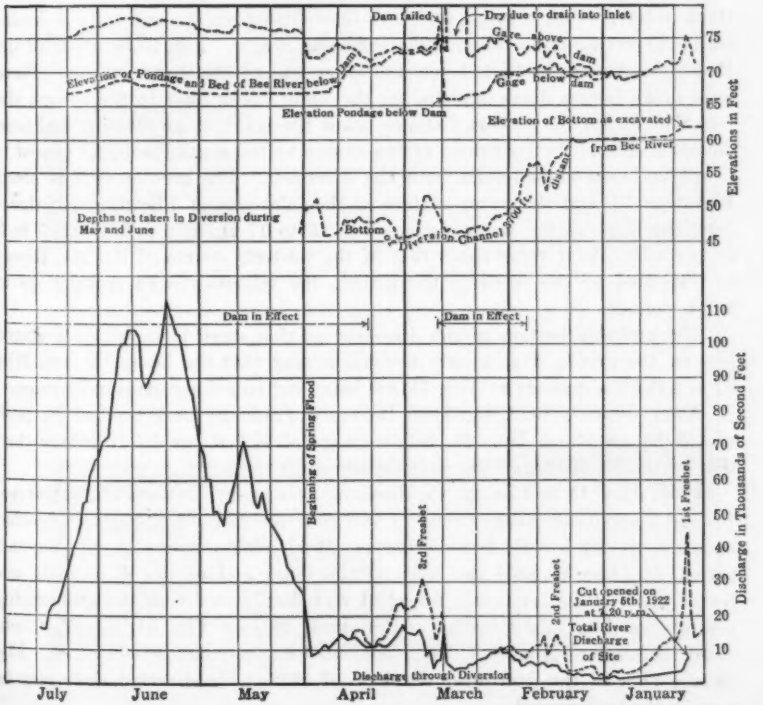


FIG. 10.

and 2 may be seen to unite in the background. Fig. 12 is a view at the south end of the dam, showing where it nearly failed on May 3. Fig. 13 is a view up stream from the meter station on May 3, and shows the effects of erosion on the spoil banks of Channels 1 and 2.

On May 6 and 7, the dam was menaced and was saved only by dumping trainloads of heavy rock along its face. Inlet conditions rapidly improved with the increasing discharges, reaching 42 400 sec-ft. on May 7, 56 800 sec-ft. on May 13, and 71 200 sec-ft. on May 18. It is noteworthy that the gauge held steady within 1.1 ft., while the diversion channel increased in capacity from 28 400 to 71 200 sec-ft., thus showing that erosion was taking place as rapidly as the increasing volumes became available.

An inspection was made in a motor launch along the diversion, for 6 miles below the dam, on May 27, and showed the excellent inlet conditions illustrated in Fig. 14, and a river, 400 to 600 ft. wide, on a straight course, overflowing its banks on both sides into the thick brush. However, it was disappointing not to be able to proceed farther because of sand-bars that had filled the deep arroyo at the bend shown at *a* on Fig. 5. From 50 000 to 60 000 sec-ft. was flowing into the inlet, and no ground was visible below it; a 3-in. overflow at, say, a velocity of 2 ft. per sec., along the 12 miles of banks (both sides), would account for more than 30 000 sec-ft., and leave a diminishing 20 000 sec-ft. to follow the overflowing arroyo. Fig. 15 is a view of the Pescadero Arroyo in 1921, below the lower end of the diversion cut. There were many beaver dams impounding the water. The kink in the arroyo, the fact that except for a short distance below the point, *a*, no clearing had been done, as well as the curvature of the arroyo to the north, probably caused it to fill with the eroded soils from the diversion. The ground at *a* is about Elevation 59, and the water surface at the inlet was at Elevation 76; therefore, the slope of the water surface was 15 to 17 ft. in 6 miles, or 2.5 to 3 ft. per mile, about three times that of the westerly course of the Bee River; as estimated by the speed of the launch, the velocities were probably 10 to 15 ft. per sec.

The probable bottom of the diversion at this stage is shown as a dotted line on the profile, Fig. 4, and it would appear that the hump between Mile 5½ and the low country at Mile 7½ was being cut in a down-stream direction.

After June 1, 1922, the flood increased gradually and reached its peak of 112 000 sec-ft. on June 10, with a gauge of 77.7 ft., or 1.7 ft. higher than that when the discharge was only 28 400 sec-ft.

From June 18 to June 23, the discharge was again in excess of 100 000 sec-ft., and a maximum gauge height of 78.0 was recorded, 0.2 ft. higher than that on June 10, for a little less discharge. Much rock was dumped during this period, in order to hold the levee at the south end of the dam, as it was feared that if the levee was breached and deeply undercut, back sloughing toward the dam might occur. After June 24 (see Fig. 4), the flood subsided, and before August 1, 1922, the flow was less than 10 000 sec-ft. Fig. 16 is a view of the dam after the flood of 1922. The diverted river may be seen in the background.



FIG. 11.—DIVERSION CHANNEL DOWN STREAM FROM METER STATION, MAY 3, 1922. CHANNEL CARRYING 29 560 SEC-FT.



FIG. 12.—SOUTH END OF DAM, SHOWING NEAR FAILURE ON MAY 3, 1922.



FIG. 13.—DIVERSION CHANNEL UP STREAM FROM METER STATION, MAY 3, 1922, SHOWING EFFECTS OF EROSION ON SPOIL BANKS.

Fig. 14 shows the changing conditions at the inlet to the diversion on the dates given, from January to June, 1922. The principal features are: (1) the difference between the widths of the river bed and the diversion channel on January 10, as contrasted with the differences in May and June; (2) the formation of silt bars in the river and across the inlet of the channel; the silt bar across the inlet of the channel, and also a part of the spoil excavated from the river bed, remained until the discharge exceeded 70 000 sec-ft.; and (3) the direction of the flow into the inlet channel, which was nearly reversed up to the time of the flood, when the making of the dam into a massive rock-fill caused the flow into the inlet to be more direct.

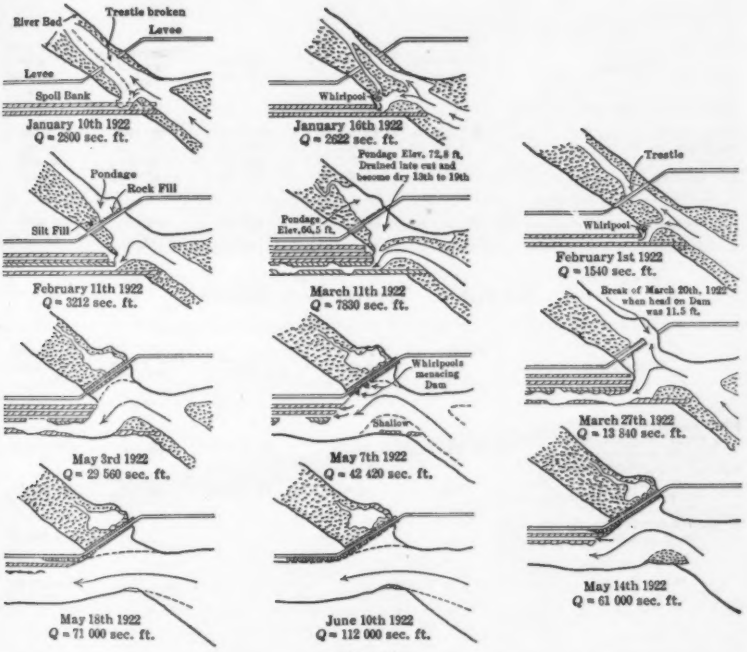


FIG. 14.

The granite rock was quarried at Andrade, Calif., along vertical faces of a developed quarry, 50 000 to 250 000 cu. yd. of rock being blasted at a time. The rock was loaded by steam shovel on automatic air-dump side-tipping railroad cars of 16 cu. yd. capacity, which were hauled to the levees and dam. The distance from the quarry to the dam is about 28 miles.

COST OF THE DIVERSION

A detailed classification of costs was outlined before the work was started and Table 5 is taken from the report of dragline operations. In this report, the costs of operation, repairs, and delays, of the dredges can be compared,

together with the yardage excavated and the unit costs. These prices include the cost of gasoline and its haulage for 7½ miles.

As much as 1 720 cu. yd. per shift of 7½ hours was attained by these dredges, and 1 100 to 1 500 cu. yd. per machine per shift was common.

An operator's activity chart was plotted daily for the inspection of the dredge crews, and this had much to do with stimulating rivalry for maintaining a high yardage output. A similar chart was kept daily showing the pile-driving operation on the trestle. The trestle cost record is given in Table 6.

TABLE 5.

Dredge.	Operating cost.	Repair cost.	Delays cost.	Excavation, in cubic yards.	Unit cost of dredge, operation only.
Bucyrus No. 1	$12 004.88	$2 331.40	$883.07	239 984	$0.0500
Bucyrus No. 2	9 431.83	1 866.98	256.22	192 345	0.0490
Bucyrus No. 3	8 088.08	1 747.11	248.89	177 100	0.0456
Monighan	4 648.05	1 431.03	274.05	47 664	0.0975
Total	$34 172.84	$7 376.52	$1 662.23	657 093	$0.0520

TABLE 6.—TRESTLE COST RECORD.

Description.	Number of piles driven.	Total penetration, in feet.	Cost of pile-driving.	Driving cost per pile.	Total cost of trestle built.
First Construction: 1 232-ft. substructure, 837-ft. superstructure	352	8 671	$2 397.12	$6.812	$19 904.91
Second Construction: 700-ft. substructure, 1 080-ft. superstructure	199	5 877	$1 972.46	$9.912	$21 573.28
Third Construction: Extension of north end and wing piling	....	....	.....	.....	$18 592.42
Total	....	....	.....	.....	$60 070.61

Tables 7 and 8 show the classification of the entire cost of completing the diversion. Table 7 gives the expenditures to March 15, 1922, construction operations having been suspended on March 1 to await the increase in the river flow. Table 8 shows the subsequent expenditures to the end of June, making the total cost $413 320.

In the excavation costs (Table 7), $10 110.55 is included for clearing; $3 175.00 for aerial survey; $34 172.84 for operations, repairs, delays, and fuel of all dredges; $26 283.71 for depreciation of dredges; $6 570.93 for depreciation of camps and roads; $17 100.56 for assembling, moving, and dismantling the dredges, camp construction, road, telephones, cableway, metering, tools, supplies, auto-vehicle operations, insurance, overhead, and engineering.

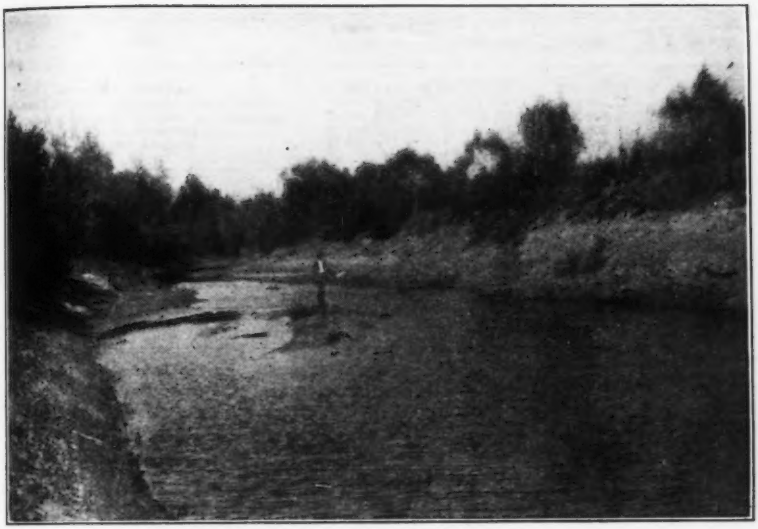


FIG. 15.—PESCADERO ARROYO IN 1921, BELOW LOWER END OF DIVERSION CUT.



FIG. 16.—VIEW OF DAM AFTER FLOOD OF 1922. DIVERTED RIVER IN BACKGROUND.

TABLE 7.—CLASSIFICATION OF COSTS OF DIVERSION TO MARCH 15, 1922.

Item.	Quantity.	Unit cost.	Cost.	Total cost.
East Channel, excavation	451 637.3 cu. yd.	$16.41	$74 117.94	.....
East Channel Extension, excavation	8 789.6 "	10.95	962.72	.....
Deepening East Channel, excavation	38 091.9 "	16.33	6 223.49	.....
Central Channel, excavation	121 136.5 "	10.41	12 605.32	.....
West Channel, excavation	37 437.4 "	9.36	3 504.12	.....
(1) Total channel excavation to March 15, 1922	657 092.7 cu. yd.	$14.82	$97 413.59	$97 413.59
Trestle, first construction	88 bents, 837-ft. superstructure	.....	$19 904.91	.....
" second construction	51 bents, 1 080-ft. superstructure	.....	21 573.28	.....
Dam, built from trestle	19 024 cu. yd., rock	$1.05	20 288.94	.....
	26 248 cu. yd., silt	0.105	4 042.87	.....
(2) Total for damming river to March 15, 1922	.....	.....	$65 810.00	$65 810.00
Clearing on north bank	74.52 acres	.....	$2 680.70	.....
Levee on north bank of river	68 346.3 cu. yd.	$0.30	20 641.89	.....
Levee west of channels	6 616.3 "	0.51	3 379.32	.....
(3) Total for overflow protection to March 15, 1922	74 962.6 cu. yd.	$0.316	$26 701.91	$26 701.91
Railroad, as first laid on ground	35 828 lin. ft.	$2.98	$96 843.41	.....
Transferring from ground to levee	25 000 lin. ft.	0.55	13 736.66	.....
(4) Total cost of railroad to March 15, 1922	.....	.....	$110 580.07	$110 580.07
(5) Preliminary cost to February, 1921	.....	.....	$25 180.94	$25 180.94
Total cost to March 15, 1922	.....	.....	.....	$325 686.51

TABLE 8.—ADDITIONAL COSTS, FROM MARCH 15 TO JUNE 30, 1922.

Item.	Quantity.	Unit cost.	Cost	Total cost.
Channel:				
Maintenance	.....	.....	$ 866.96	
Purchase of motor-boat	.....	.....	1 735.74	
				$2 602.70
Trestle:				
Repairs and extension at north end, owing to break of March 20, 1922; also pile wing	29 200 cu. yd. rock.	$1.14	$18 592.42	
	3 040 " " silt.	0.10	33 451.67	
Dam:			304.00	
Total for rebuilding and strengthening dam	.....	.....	.....	$52 348.09
Levee on North Bank:				
Raising	23 157 cu. yd. earth.	$0.083	$1 918.90	
Rock revetting levee	23 872 " " rock.	0.952	22 737.80	
				$24 656.70
Railroad:				
Transferring from ground to levee from March 15 to June 30, 1922	10 500 lin. ft.	$0.76	$8 026.08	
				$8 026.08
Total cost of diversion from March 15 to June 30, 1922				$87 633.57

In Table 8 is included the cost of repairing and lengthening the trestle, the strengthening of the dam, and the rock revetment which extends the entire length of the levee along the north bank.

The wages paid were, as follows:

For Dredge Operations:		
General foreman of machines	$9.60	per day
General mechanic	210.00	per month
Chief operators, one for each machine	9.00	per day
Operators	1.00	per hour
Engineman	0.70	" "
Groundman	0.375	" "
For Trestle Construction:		
Pile-driver foreman	$1.00	per hour
Enginemen	0.875	" "
Pile-driver men and helpers	0.75	" "
Bridge carpenters	0.75	" "
Firemen	0.50	" "
Laborers	0.30	" "
For Miscellaneous Work:		
Speeder drivers	$144 and $102	per month
Auto-vehicle mechanic	0.70	per hour
Boatmen	0.30	" "
Laborers, first-class	0.30	" "
Laborers, second-class	0.25	" "
Engineering and Clerical:		
Instrumentman	$204	per month
Rodmen and chainmen	$132 and 102	" "
Costkeeper	174	" "
Storeman	138	" "
Timekeeper	138	" "
Commissary:		
Cooks	$102 and $72	per month
Flunkies	54	" "

There was deducted from all labor $1 per day for meals furnished at the Commissary.

The Value of the Diversion

In studying the economy of the diversion, the cost of all the work that would have been immediately necessary to the Saiz and the Volcano Lake Levees, had the diversion not been accomplished, must be considered.

A part of this work would have been the construction of a railroad along the Saiz Levee for revetment purposes, all of which would have cost nearly $1 000 000. Also, it would have been necessary to heighten and revet the Volcano Lake Levee, and this would have cost $500 000. To these costs must be added the expected annual cost of repairing and raising these levees and the railroads, for the expected life of the diversion. If the sum of these costs is compared with that of the diversion and its expected maintenance, an idea may be gained of the direct value of the diversion.

Indirect assets will result because an annually increasing flood menace to the welfare of a great irrigation project has been alleviated for a number of years, or until permanent relief can be assured by the construction of the Boulder Canyon Dam. The diversion tends to promote confidence that will be beneficial in many ways. New capital has already been invested, and crops have been insured against floods in Baja California for the first time.

The period that the river can flow into the Pescadero Country before the flood water again reaches the heights of 1920-21 against the Volcano Lake Levee, can be estimated as follows: The distance along the Bee River channel from the diversion point to Volcano Lake is about 24 miles and the lake level is about 36 ft. below the ground surface at the inlet of the diversion. At Mile 24 on the profile (Fig. 3), plot Elevation 40 and draw the grade line to Elevation 76 at the inlet, extending the line toward the Gulf. The ground surface along the meanders of the diverted river must become elevated to this line before it will have the same slope that it has toward Volcano Lake.

This line is 15 to 20 ft. above the ground surface beyond Mile 7 and, for a meandering course that will not be less than 40 miles long, 15 to 20 years of deposition would be required to reach a slope similar to that which has been made along the course of the Bee River since 1909. This condition may require the building of a deflecting levee to the south end of Volcano Lake, as the growing elevation up stream from Mile 24 becomes apparent, depending on the direction of the overflow streams.

A conservative estimate of the area below Elevation 40 and above Elevation 20 in Pescadero Basin is 130 000 acres. If it was possible for the river to silt only this area to Elevation 40 (the elevation of Volcano Lake), 1 300 000 acre-ft. of soil would be necessary, and this would require a flow equal to that of the last 14 years, based on the silt deposit for the past decade. It would seem, therefore, that the river could be directed into this basin for at least 15 years, before water to great depth can be stored against the Volcano Lake Levee.

Surveys extending down stream along the diverted river, made as soon as the overflowed country had become dry again, revealed a most interesting unlooked for condition as an aftermath of the flood along the diverted route.

Along the whole length for which the clearing had been taken, the overflowing river has elevated the contiguous ground from less than 1 ft. at the point of diversion, to a gradually increasing height of 13 ft. maximum at 4 to 5 miles down stream. In other words, the steeper ground slopes did not result in a deepened channel on the new course, but an elevated country was built, approximating the gradients on the former westerly course. The new ground surface, elevated by deposition during the 10 weeks of the 1922 flood, is shown on Fig. 4; the diagram also shows an intensified slope that now exists between Mile 5 and Mile 10, due to a difference in elevation of about 30 ft.

This newly built country has a maximum elevation along the banks of the diverted river and slopes to nothing at about 3 000 to 4 000 ft. from the river A cross-section would show an excellent demonstration of a débris-cone formation produced by a turbid river.

Conclusion

The Engineering Department of the Imperial Irrigation District, under the direction of F. N. Cronholm, M. Am. Soc. C. E., made the studies and surveys that determined the diversion.

The work during 1922 was conducted by Ray S. Carberry, M. Am. Soc. C. E., Chief Engineer of the Imperial Irrigation District, to whom the writer extends his appreciation for the opportunities given to follow the progress of the diversion, and to place this description of it on record before the Society. F. E. Higley, Superintendent of Construction, is to be commended for his untiring efforts in maintaining the levees and the dam in an efficient condition during the flood. The diversion might not have been completed during the first flood, if it had not been for Mr. Higley's ceaseless vigilance during the freshets and the 10 weeks' duration of the annual flood. E. E. Kiefer, Chief Cost Accountant, outlined and supervised the collection and classification of costs.

DISCUSSION

PAUL M. ENTENMANN,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper is a valuable record of an interesting river-control structure on the lower reaches of the Colorado, and is particularly noteworthy for its presentation of the data of the preliminary investigations which determined the type and location of the diversion. In a region where rule-of-thumb methods, guess, and snap judgment often supplant critical scientific research, it is gratifying to find that the work was planned with some care. Although this diversion was developed exactly as had been planned, the writer thinks that Mr. Rothery is too optimistic in his conclusions with respect to the feasibility of directing the river into new channels. Explaining the capacity of the river to erode its own bed, he says:

"With such rapid erosion, provided increasing volumes of water are available to the diverted course, any channel can be quickly enlarged so that the entire river flow will soon follow the new course, especially if its old direction on flatter gradients is effectively blocked."

Too little emphasis is placed on the necessity of closing the old channel in order to assure success; and the importance of the rock-fill dam across the old channel into Volcano Lake and of the levee which was built along the north bank of the river, is insufficiently stressed. In considering the success of this diversion, it should be understood clearly that the river had no choice of direction. Its flow in any other direction than that of the diversion was most effectively blocked; and factors other than that of fall contributed to the success of this channel change. Where the river has a choice of either of two channels, one on a flat gradient in the general direction of its flow, and the other on a steeper gradient at an angle to this direction, the kinetic energy of the stream may offset the theoretical possibilities of greater velocities on the steeper gradient and continue the stream in its old channel, especially as the eddy currents created at the point of diversion immediately cause the deposition of silt and the building of bars across the inlet of the new channel.

Stating this differently, consider the conditions at the point of the diversion. In the direction of the old channel of flatter gradients, the stream has an initial velocity which may be as high as 9 ft. per sec.; in the direction of the diversion, the initial velocity may be zero. In fact, in Fig. 14, Mr. Rothery shows one inlet condition where the stream actually had to reverse itself to get into the new channel.

The Colorado, an alluvial stream flowing in its own alluvium, has such a high silt content that at all stages it is chocolate colored. As the silt has a specific gravity of 2.65, it is held in suspension by the internal work of the moving stream; and whenever the river is ponded, or its velocity decreased, deposition of the silt takes place very rapidly.

Where no obstruction is placed across the old channel, permitting the major portion of the stream flow to continue in the old direction at the time the diversion channel is first opened, the change in direction of that

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part of the river which leaves the old channel produces eddy currents; and there is an area between the eddy current and the main current where the stream has zero velocity. In this area, sand-bars are quickly formed and, in general, the bar begins at the mouth of the diversion near its up-stream bank. Further reversal of current follows the formation of the bar, with accompanying increase in deposition of silt and the prolongation of the bar down stream, until it extends entirely across the mouth of the new channel. It is evident that such an obstruction across the inlet of the diversion channel seriously affects the chances of successfully diverting the stream in the new direction when the old channel remains unobstructed.

The writer is familiar with six attempts to divert the Colorado River into a new channel, no dam being used to block the old channel; and five of these attempted changes were failures. It is the purpose of this discussion to describe the cut-off that succeeded and to note the influences that caused the failures.

About 100 miles north of the delta is a low-lying, crescent-shaped valley, 30 miles long and approximately 8 miles wide—the Palo Verde Valley—contiguous to the west bank of the Colorado River. The valley lies below the flood level of the river and would be subject to inundation from the annual June floods, if it was not protected by a levee system. The land is highly cultivated, being served by a gravity irrigation system taking its supply from the Colorado at the north end of the valley. Its protection system is an earth levee with a top width of 12 ft. and side slopes of 3 on 1 on the river side and 2 on 1 on the land side. The top of the levee is about 3 ft. above flood height. With the exception of two places, each about ½ mile long, where it is heavily blanketed with rock rip-rap, at least 10 ft. thick, the levee is unprotected from erosion; and, therefore, it has been built a considerable distance back from the normal channel of the river, so that it functions chiefly as a dam to hold back the ponded flood waters, and not as a guide to determine the location of the channel. During recent years, owing to the tendency of the river to build up its own bed and thus increase the slope toward the levee, the channel has moved dangerously close to the levee at four places; and during the floods of 1921 and 1922 the barrier was breached and large areas of the valley were submerged. Although the value of the crops destroyed in these floods, and the resulting depreciation of land values, far exceeds the cost of protecting the levee with rock rip-rap, the settlers have been reluctant to finance the work of blanketing the levee with rock and of placing a railroad on it.

As the necessity of preventing the river from impinging on the levee is urgent, and is recognized by the settlers, many schemes for directing the channel away from the levee during floods have been proposed and tried. Every cross-roads theorist has some pet panacea for saving the valley; and the channel cut-off is the cure-all most often suggested. There is a theory, which has become almost a folk-lore tradition among the settlers, that if the river was "straightened", all flood dangers would vanish.

The river meanders along the 30 miles of valley in wide sweeping bends which increase the length measured along its course to more than 50 miles. These bends are convex toward the levee at four places, and at these places attempts have been made to divert the channel into a cut-off across the bend.

The Olive Lake Bend offered the best conditions for the success of an attempt to divert the river flow across it. It is a wide horseshoe-shaped bend, 8 miles around the curve, and only 2 miles across it. As the fall of the water surface around the bend is about 1 ft. per mile, the fall across it from water surface to water surface is 4 ft. per mile, but the ground slope for the first mile is steeper. At every spring flood, the river in overflowing its banks deposits most of its silt near its banks, the up-building of the ground surface subject to overflow being a maximum at the edge of the channel and diminishing to zero at a distance of 3 000 to 4 000 ft., the typical débris-cone formation. Therefore, at this bend, the river flowed around the rim of a saucer-shaped area, and the profile on the line of the proposed cut-off showed a fall of approximately 5 ft. for the first mile, with flatter slopes for the remaining distance.

Above the head of the bend, the river flows southward, turning to the west sharply at a right angle as it enters the bend. The diversion was at the apex of this angle, and the cut-off was oriented so that, in effect, it would be a continuation of the southerly flowing channel above the bend.

In the early spring of 1920, a right of way, 300 ft. wide, for the proposed cut-off was cleared through the heavy brush, and a cut, with a bottom width of 50 ft., side slopes of 1½ on 1, and approximately 1 ft. below the low-water level, was excavated with teams and Fresno scrapers. The excavation could not be made deeper because ground-water conditions limited the depths at which teams could work, and no mechanical equipment was available. During low-water periods, when the flow is between 5 000 and 8 000 sec-ft., the river surface at this place is 5 ft. below the top of its banks; and as its maximum flood level is about 10 ft. higher, it overflows these banks to a depth of 5 ft. during the June freshet.

The cut was opened on April 25, 1920, when the river flow reached 30 000 sec-ft., and when the head on the cut was about 5 ft. The effect was disappointing. The channel above the diversion began to develop a bend to the east, so that, instead of flowing directly into the cut as had been planned, it made an angle with the direction of the new channel, which increased to nearly 90° as the bend above enlarged. From April 25 to May 2, the discharge decreased to 17 000 sec-ft., with no appreciable effect on the cut; but the poor inlet condition, due to the unfortunate channel change above it, was aggravated by the formation of a silt bar across the mouth of the diversion, which threatened its early failure. After May 2, the river discharge increased steadily to the peak of 190 000 sec-ft. on June 7, and this discharge, with consequent increased head on the diversion, together with the lavish use of explosives on the silt bar across its mouth, increased the discharge through the new channel; but it was not until the river topped its banks on May 17, when the total discharge had reached 50 000 sec-ft., that the anticipated

erosion of the bottom and banks of the diversion began. At the mouth, the cut widened and deepened rapidly, and the increased flow through the new channel indicated a possible success; but the eroded material, transported by the water in the new channel at its gradient of 4 ft. per mile, was carried to the end of the 2-mile diversion where it re-entered the old channel, flowing at lessened velocity on a gradient of 1 ft. per mile. The consequent retardation of the current resulted in the immediate deposition of silt. In fact, within 2 weeks, a perfect delta cone had formed at the outlet, with the water flowing over it in a wide thin sheet instead of through a confined deep channel.

Increasing discharges in the river had little effect on this delta formation. The cone continued to build itself higher, and farther up-stream, with the result that the gradient of the cut-off was flattened and no further erosion of the banks or bottom took place. When the flood reached its peak on June 7, with a discharge of 190 000 sec-ft., the diversion carried approximately 48 000 sec-ft., including the water which overflowed the banks of the new channel. The channel had eroded to a width of 300 ft., with an average depth of less than 4 ft. Continuous effort was required to keep it open.

The river in flood carries enormous quantities of drift—dead tree trunks, branches, logs, and a large quantity of green vegetation from the eroded banks, chiefly young willows and cottonwoods—and this drift fouled everywhere throughout the channel and formed sand-bars behind it, with resultant decreased discharge capacity. A patrol in a motor launch and several row-boats worked continuously with explosives to remove snagged drift; explosives were also used in an attempt to aid the erosion of the bottom.

After June 7, the discharge continued to decrease steadily until July 1, when it was 50 000 sec-ft. During this reduction in quantity no change in the carrying capacity of the diversion took place; but the river above the inlet developed a sand-bar on the east side which closed the bend that had formed when the cut-off was opened, and caused the main channel to head straight into the diversion. Strenuous efforts were then made to keep the cut open. Explosives were used freely to remove all drift, and an opening through the delta cone at the outlet was made by exploding large quantities of dynamite, in order to confine the stream to a narrow channel. A recession of the bottom took place, and within four days had worked back to the inlet of the diversion, lowering the water level of the river there nearly 2 ft., sufficient to reduce the flow in the direction of the old channel to the point where this channel was rapidly silted in. On July 18, when the discharge had diminished to 30 000 sec-ft., the entire flow passed through the diversion.

It may be of interest to note in what respect the conditions that determined the success of the delta diversion described by Mr. Rothery, differed from those of the Olive Lake Cut-Off. The ground-surface gradients were almost the same; but, in the delta diversion, the ground slope beyond the end of the excavated channels is steeper than the gradient to which the channels were excavated, whereas at Olive Lake, the diversion emptied into a flatter basin. At the delta, increasing quantities of water were forced through the diversion as the discharge of the river increased, because the dam across the old channel

closed all other routes. At Olive Lake, the new channel never carried more than 50 000 sec-ft. (including the quantity that overflowed the banks). The change in direction at the delta was effected by the rising river, and was complete at the peak of the flood on June 7, whereas the change at Olive Lake took place on a falling river, and became established when the discharge had dropped to the point where the river was confined completely within its banks.

The writer believes that the change at Olive Lake was the result of luck and the freakishness of the river. If the channel above the inlet to the cut-off had continued around the curve, making an angle with the direction of the diversion, the mouth of the new channel would probably have been silted in on the falling river and the flow would have continued around the old channel. It was the change in the direction of the channel above the inlet (a change which was purely accidental and in no way related to the conditions in the cut-off channel), which directed a mass of water, with a cross-section 500 ft. wide and 15 ft. deep, moving with a velocity of 7 ft. per sec., straight into the cut, that finally diverted the river across the bend.

Previous attempts to divert the river across the Olive Lake Bend had been made in 1917 and in 1919. In both attempts, an old overflow channel was cleared, and explosives were used to help the erosive action of the rising river. The channel eroded to a width of 600 ft. with a depth of 6 ft., but silted in again when the river dropped to its low-stage level.

Another attempt at diversion was made in 1919, across Raab's Bend, where an old channel on the Arizona side, known as the Ehrenburg Cut-Off, carries part of the river flow at all stages. It has a slope of 3 ft. per mile, and the channel around the bend slopes about 1.25 ft. per mile. Two jetties, 400 and 700 ft. long, respectively, with the longer one 500 ft. down stream from the shorter, were built from the west bank into the main channel of the river at an angle of 135 degrees. They were just above the entrance to the Ehrenburg Cut-Off on the opposite bank, and were constructed in such a position that the current of the river, if deflected along the face of the jetties, would head straight into the cut-off channel. The jetties were built of 3-pile bents, driven with a penetration of 20 ft. below low-water bottom; the bents were 15 ft. from center to center, capped with 10 by 10-in. timbers, connected with three lines of 8 by 10-in. stringers, and laced in both directions with 3 by 12-in. bracing, all bolted. The jetties were filled with brush bundles, about 3 ft. in diameter and 20 ft. long, made of young willows 3 in. in diameter at the butt. These bundles were tied with heavy wire and lashed to the piles with light cables. These jetties were built in order to deflect the main channel through the cut-off across the bend, and were complete failures.

The theory which dictated the construction of these jetties is fallacious. It was assumed that the current of the river striking an obstruction placed at a wide angle with the bank, would be deflected along the face of the obstruction and continue in this direction. Even if the face of the jetties could be made very smooth and the alignment curved to afford an easy transition, such a deflection might not take place. What actually happens is this: The stream striking the jetty with a velocity of 9 ft. per sec., loses this velocity at

the up-stream face of the jetty by converting its velocity head into static head, so that there is a difference of head between the up-stream and down-stream faces of the jetty of 12 to 14 in. in a distance of 15 ft. This steep slope around the end of the jetty has the effect of turning the current back toward the down-stream face in the opposite direction from that in which the jetty was designed to send it. The velocity at the end of each jetty is very high. Velocities of 18 ft. per sec. were measured when the normal channel current was moving at 9 ft. per sec.

Other feeble attempts to divert the channel into cut-offs across bends have been made at Palo Verde, but all have failed because it was assumed that, if a small channel was cut across a bend, the river would abandon its normal course (without an obstruction across it) to flow down the steeper slope, where it would erode a channel of sufficient capacity to carry the flood flow. The formation of a delta at the end of the cut-off channel, the obstruction of silt bars across the mouth, the accumulation of drift in the new channel, and the failure to construct the channels deep enough to carry the entire low-stage flow, all help to keep the flow in the old course.

The diversion of the river through the Olive Lake Cut-Off was calamitous in its effects on the further development of Palo Verde Valley. The organization in control of the protection system of the levee rejected the policy of blanketing the levee with rock, and made further attempts at channel changes. In 1921, the river breached the levee and flooded about one-fifth of the valley; and again, in 1922, another break in the levee, caused by the river impinging against it, brought a disastrous flood which submerged about one-third of the most richly cultivated area of the valley.

In the spring of 1923, further attempts at river diversion were made at Palo Verde. Three wide sweeping bends in the river, known locally as Raab, Hauser, and Casad Bends, are convex toward the levee and approach it closely; and at each of these bends an attempt was made to divert the river into channels across the bend. The conditions effecting diversion are, in general, the same at each of the bends. The length of the channel around the bend is 4 miles; the distance across it is 2 miles; the fall of the water surface around the bend is 1 ft. per mile and across it, from water surface to water surface, is 2 ft. per mile. A right of way, 500 ft. wide, was cleared across each bend, and a channel, 50 ft. wide, 5 ft. below low-water level, with side slopes of 1 on 1, was excavated with dragline excavators. The cuts cost approximately $75 000 each, not including maintenance costs. They were opened early in May, as the river began to rise with the approach of the annual summer flood.

No high peaks occurred in the June flood of 1923. The maximum discharge of the river did not exceed 100 000 sec-ft. (as compared with high peaks of twice this quantity in previous years), and yet none of these diversion channels carried the total discharge. At the Raab Bend, the cut-off channel carried less than 20% of the flood flow, the remainder continuing around the old channel. At the Hauser Bend, the flow was equally divided between the diversion and the former channel. At the Casad Bend, the cut-off

channel carried 80% of the flood flow and, at present, it seems probable that at the low stage the entire river will be diverted through the cut-off channel. However, unless an obstruction is placed across the old channel, there is no assurance that the next flood will follow the diverted route, especially if the peak should reach 200 000 sec-ft.

Present conditions forecast the failure of the cut-off across the Raab Bend and indicate that the cut-off at the Hauser Bend will probably divert only a part of the low-stage flow. As the purpose of these cut-off channels is to divert the entire flood flow, and as a channel of sufficient capacity to carry the usual peak flood of the river has not been eroded along the diverted route of any of these diversions, the levee is not safe from a possible breach due to widening of the bends during flood stages of the river unless additional construction effectively blocking the old channel is completed and the entire flow of the river forced into the cut-off channels.

S. L. Rothery,* Assoc. M. Am. Soc. C. E.—Mr. Entenmann's discussion is valuable from two viewpoints. It is a record of the influences pertinent to the success of a diversion known as the Olive Lake Cut-Off; and it is a warning against the futility of costly attempts at diversion by incorrect or incomplete measures, that are the result of insufficient engineering investigation, or of ambitious proposals that have not had the scrutiny of competent engineers familiar with the river to be diverted.

The work undertaken to cause a diversion on the delta of the Colorado was no desultory effort. Existing maps were studied, and new surveys to determine various grades and channel shapes were made; cost estimates and methods of doing the work were prepared; and the feasibility of erosion was studied and discussed. When the work was started, it was actively carried out to completion.

The profiles, Figs. 3 and 4, showed that there was no obstruction to the free discharge of any maximum volumes of flow into the Pescadero country. The doubtful features were, would erosion occur as expected, to keep the main discharge confined in a channel, or, would the small excavated channel become choked with driftwood and silt?

Adverse conditions would have caused failure as soon as the overflowing spillway along the five miles of the south bank of Bee River had become elevated, to the height of the top of the dam, by deposition due to the dense vegetation there. Such conditions could have occurred in a few days and the dam would have failed by overtopping.

Erosion did occur as the quantity of water increased, as illustrated by the reference, on page 1420, to the capacity of the river for erosion in its own alluvium. Proof of this is that, after the river channel was filled to overflowing with 30 000 sec-ft., the gauge never raised more than 1.9 ft.

Mr. Entenmann's exception to this illustration does not apply exactly at the Pescadero Diversion, as there was no ponding in the 4 to 6 miles of channel to retard erosional velocity. At the outlet end, beyond the ridge of the south bank of Bee River, the whole Pescadero country is available for the outflowing water. This country is too wide and has too great a slope toward the

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Gulf to permit a pondage. Dense vegetation has caused heavy deposition and ground elevation in this country. The ground, however, may be built much higher, even above the present tree-tops, before its elevation would approximate that of the banks of Bee River up stream from the dam.

In a stream having an eroding channel, in which the quantity of flow is increased ten to twenty times, the kinetic energy gradually increases as the erosion of the channel gradually increases to the proportions of a river, the

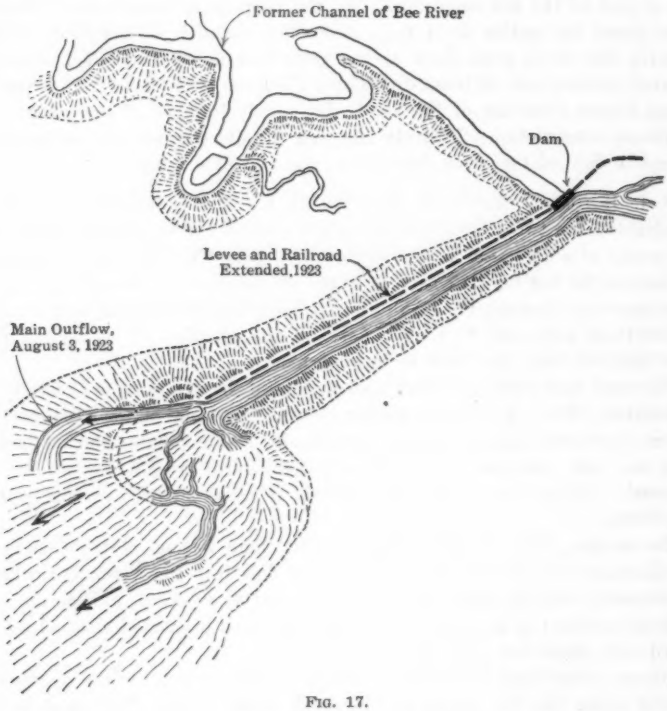


FIG. 17.

bed of which is far below the surface of the ground. The moving flood is confined and guided in that channel, so that where a dam is built the kinetic energy is controlled at the diversion inlet, exactly as at any natural bend on the river.

Without a dam, the flow would be divided and it would be impossible to predict which channel would receive the major part.

The rock-fill dam and its maintenance is an important feature to assure success, and it represented 28% of the total cost of the Pescadero Diversion. Without a dam, increasing quantities of water are not available to the diversion inlet of a small excavated channel across a river bend, because the water passes around the bend with increasing velocities and depths of scour, thus

leaving the cut-off channel somewhat like an elevated canal in which the velocity is not sufficient to produce erosion, and driftwood forms log jams which cause silt deposition and the probable failure of the cut-off.

At the confluence of the small stream flowing through the cut-off with the larger stream flowing around the bend, eddies are caused and changes in velocity and direction are produced. As a result, a delta cone of silt is likely to be formed at the outlet, with an overflowing thin sheet of a water instead of a confined deep channel, as has been described by Mr. Entenmann.

Fig. 17 shows the present slopes for 6 to 10 miles along the diverted course and the new Pescadero Levee and railroad that was extended southerly along the west bank from the dam. This levee was built prior to the 1923 flood in order to prevent overflow toward the channel of the Bee River Bend. Fig. 18 is a profile showing the height of this levee and the high-water elevations of the 1923 flood which did not exceed 100 000 sec-ft. The flood was of

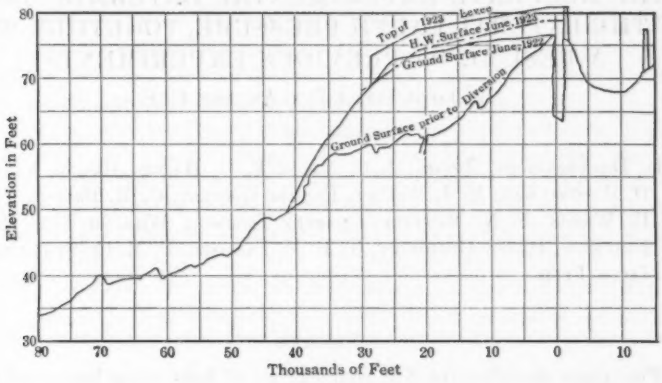


FIG. 18.

normal duration and undoubtedly produced much silt deposition in the dense foliage below the south end of the diversion. At the dam the gauge increase was 3.75 ft., and at the south end of the Pescadero Levee, the increase was 1.0 ft., subsequent to the first freshet of 30 000 sec-ft. on May 10, thus showing that a free discharge existed at the outlet end. The peak of the flood occurred on June 9 and again on June 26, 1923.

Between July 1 and August 3, 1923, the water surface was lowered 5.5 ft. at the dam and 7.9 ft. at the south end of the levee, with a decreasing discharge of from 63 000 sec-ft. to 24 300 sec-ft. This is evidence that recession is taking place.

Also, on August 3, 1923, the river was flowing in a southwesterly direction down the steep slope below the south end of the levee, with velocities of about 8 ft. per sec., and was confined in a deep channel with banks about 10 ft. high, as shown in Fig. 17.

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Paper No. 1529

LATERAL EARTH PRESSURE: THE ACCURATE EXPERIMENTAL DETERMINATION OF THE LATERAL EARTH PRESSURE, TOGETHER WITH A RESUME OF PREVIOUS EXPERIMENTS*

By Jacob Feld,† Jun. Am. Soc. C. E.

With Discussion by Messrs. J. C. Meem, E. G. Haines, George Paaswell, D. B. Steinman, M. J. McPike, Edward Godfrey, G. M. Braune, Thomas H. Wiggin, F. N. Menefee, Charles Terzaghi, William Cain, Milo S. Ketchum, Henry Goldmark, T. E. N. Fargher, P. M. Crosthwaite, and Jacob Feld.

Synopsis

This paper describes the first complete set of tests using large-sized apparatus for measuring directly by means of platform scales the components of the active pressure on a model wall, 6 ft. high and 5 ft. wide. The only restraints on the wall were the measuring devices. Tests with sand under various conditions, using vertical and battered walls backed with wood, glass, and sheet iron, indicate the reliability of the Poncelet theory of the maximum wedge causing oblique pressure. Tests with walls battered in both directions show that the direction of the resultant pressure is always inclined to the normal at an angle equal to the angle of friction on the back of the wall. The effect of surcharge does not agree with the ordinary theoretical assumptions. Settling and variations in temperature cause remarkable changes in the lateral pressure.

The discussion of recent experimental data, together with a brief outline of the earlier experiments, furnishes only the minimum information necessary for a comparison between the former and the writer's results. The methods of analyzing the earth-pressure problem with the restrictions on each method, are the result of a study of more than 650 references, covering practically every-

* Presented at the meeting of March 7, 1923.

† Brooklyn, N. Y.

thing written on the subject of earth pressure. The suggested practical formulas for the determination of the lateral pressure in retaining wall design are based on the experimental data.

Appreciation

The experimental determination of lateral earth pressure was assigned to the writer when he was appointed a Baldwin Fellow in Civil Engineering at the University of Cincinnati, during July, 1919. The problem has been studied by the Civil Engineering Department of the University since 1912, when G. M. Braune, M. Am. Soc. C. E., then Professor in Civil Engineering, conceived the idea of a large size apparatus to test the lateral pressure of soils. With the co-operation of Professor C. C. Meyers, of the Department of Industrial Engineering, a preliminary design was made, based on that of Mueller-Breslau.* During 1916, B. H. Wolfkoetter, Fellow in Civil Engineering, developed the details for this design and the writer, under the advisership of Professor Braune, completed the present design. Suggestions were made by several men, who are mentioned in the description of the apparatus. To Professor Braune is also due the credit of completing the apparatus, obtaining the necessary funds, etc. E. K. Ruth, M. Am. Soc. C. E., checked the plans and computations of the design, Mr. Gerald Fitzgerald aided the writer in assembling the apparatus and in the preliminary tests. The Stacey Brothers Gas Construction Company, of Elmwood, Ohio, furnished the measuring parts of the apparatus.

The writer wishes to express his gratitude to Professor Braune, who, as adviser in 1919-21, interested him in the subject and supervised the work of designing, constructing, and erecting the apparatus. He is also indebted to his later adviser, E. D. Gilman, M. Am. Soc. C. E., who has made many suggestions that have aided in bringing the experimental and theoretical parts of this work to a successful end. Thanks are also due to Professor L. T. More, Dean of the Graduate School of the University of Cincinnati, and D. B. Steinman and F. E. Schmitt, Members, Am. Soc. C. E., for advice and encouragement, without which the writer would probably not have completed the work.

Notation

Study of the earth-pressure problem has been hindered greatly by lack of uniformity in notation and in definitions. It is hoped that the following definitions of the factors entering the determination of the lateral pressure of granular materials, will lead to standardization. As far as possible, reference is made to the originator of each word or idea defined. The necessity for such detail in definition will be understood from a comparison of the standard books on the subject. There seems to be a hopeless confusion of nomenclature as well as a lack of information on the history and development of the theories. The following notation will be used:

* H. Mueller-Breslau, "Erddruck auf Stützmauern" (1906).

Distances:

b = width of wall, measured horizontally.
h = height of fill, measured vertically.
l = length of wall, measured along the back.
s = height of equivalent fill, in case of surcharge.
x = height of resultant above base of wall.

Densities:

w = weight of wall, in pounds per cubic foot.
y = weight of fill, in pounds per cubic foot.

Coefficients:

c = coefficient of cohesion, in pounds per square foot.
f = " of friction, corresponding to natural slope.
f' = " of friction of the fill on wall.
f_1 = " of internal resistance.
f_2 = " of internal friction.

Forces:

E = total theoretical earth pressure.
P = resultant experimental earth pressure, the components being, H, horizontal; V, vertical; N, normal; and T, tangential.

Angles:

α = angle between the vertical through the heel and the back of the wall and is negative if the top of the wall overhangs the heel.
δ = angle between the resultant earth pressure and the horizontal.
ϕ = angle of natural slope or repose.
ϕ_1 = angle of internal resistance.
ϕ' = angle of friction between wall and fill.
ϵ = angle between the free earth surface and the horizontal and is negative if the surface is below the horizontal.
ω = angle between the plane of rupture and the vertical.

Coefficients:

C : of $E = \frac{1}{2} y h^2 C$, in the Poncelet theory.

$$C = \left[\frac{\cos(\phi - \alpha)}{(1+n)\cos\alpha}\right]^2 \frac{1}{\cos(\phi' + \alpha)}$$

$$n = \sqrt{\frac{\sin(\phi + \phi')\sin(\phi - \epsilon)}{\cos(\phi' + \alpha)\cos(\alpha - \epsilon)}};$$

N : of $E = \frac{1}{2} y h^2 N$, in the restricted wedge theory (E normal).

$$N = \left[\frac{\cos(\phi - \alpha)}{(1+n)\cos\alpha}\right]^2 \frac{1}{\cos\alpha};$$

$$n = \sqrt{\frac{\sin \phi \sin (\phi - \varepsilon)}{\cos \propto \cos (\propto - \varepsilon)}};$$

R : of $E = \frac{1}{2} y h^2 R$, in the Rankine theory.

$$R = \cos \varepsilon \frac{\cos \varepsilon - \sqrt{\cos^2 \varepsilon - \cos^2 \phi}}{\cos \varepsilon + \sqrt{\cos^2 \varepsilon - \cos^2 \phi}}.$$

INTRODUCTION

Natural Repose.—Any granular, semi-solid, or semi-fluid material, if unsupported, assumes a surface sloped at an angle to the horizontal; the maximum angle that a material can assume is called its natural slope, or angle of repose, which is designated by the symbol ϕ. The natural repose of liquids is horizontal. The resistance of the grains or particles of granular materials to rolling over each other tends to neutralize the effect of gravity in bringing the material to a level. At angles less than the angle of repose, a grain is held by this resistance, whereas, at angles more than the angle of repose, the resistance is less than the gravitational attraction and the particle rolls down. Woltmann, in 1799, introduced the theory that the tangent of the angle of repose must be the coefficient of friction of the granular material on itself, and, until recently, this theory has been accepted. As the angle of repose is a surface phenomenon, a plane of equilibrium on the surface, one is not justified in assuming the same amount of resistance inside the mass. Experiments have proved that the angle of friction inside the mass is not the angle of repose.

Internal Resistance.—The coefficient of friction is determined experimentally by measuring the force required to move a known mass of material resting on a plane of the same material. It is found that two coefficients may be determined, one corresponding to the force required to start the mass in motion, the other corresponding to the force required to keep the mass moving uniformly. In the first case, the force required is equal to the total resistance exerted by the mass to shearing along the plane. This resistance consists of two parts: The resistance of the particles to rolling over each other, called friction, and the surface attraction between particles, called cohesion. This coefficient is called the "coefficient of internal resistance", and the corresponding angle is the angle of internal resistance. When motion occurs, the friction acts, but the cohesion does not, so that the second coefficient found, which is smaller than the first, is called the "coefficient of internal friction", and the corresponding angle is the angle of internal friction.

Internal Friction.—It was pointed out by Boussinesq in 1883 that the formulas for the phenomena in a granular mass should be functions of the angle of internal friction rather than the angle of repose. In such phenomena, the mass is originally in a state of rest and changes to a state of non-equilibrium, either motion or unbalanced stress, whereas the angle of repose is measured after the granular material has changed from a state of non-equilibrium to a state of rest. This prediction has been fully realized in recent experiments.

Laws of Friction and Cohesion.—In 1781, Coulomb formulated the laws of friction of solid bodies and of the cohesion of bodies. He assumed that the same laws hold true for granular materials and, with only few exceptions, all writers on the subject have made this assumption:

1.—The frictional resistance on any surface is equal to a constant, that is, the coefficient of friction, times the total normal pressure on that surface.

2.—The cohesion resistance is equal to a constant times the area of the surface.

Both forces act on the surface and in a direction opposite to that of the attempted motion. Whether the cohesion resistance is a cohesion, a surface attraction between like particles, or an adhesion, a surface attraction between unlike particles, is a doubtful point. It is probably an adhesion, because each grain is coated more or less by a film of water. In dry sand, there is no cohesive force; the attraction is then between the grains of earth and the water.

Active and Passive Pressures.—If a mass of earth is assumed to be divided into two sections by a plane, the forces exerted by one mass are just neutralized by the forces exerted by the other mass. Otherwise, the plane would not be in equilibrium. Either section may be removed and replaced by a single resultant force without changing the state of equilibrium. The minimum value of this resultant force is the required strength of a wall that is to hold in place the remaining section. If this force, which is called the "active earth pressure", is increased, it will not only hold the earth in equilibrium but will also tend to move it upward. Such motion is resisted by the friction and cohesion in the mass. When the force has been increased to a value that just overcomes these resistances, and the earth is on the point of moving upward, it is called the "passive earth pressure". A better name for it would be the "passive resistance to pressure", because it is equal to the latent resistance which the mass can exert to overcome forces tending to cause upward motion. A retaining wall furnishes an example of active pressure, usually called "lateral earth pressure". Foundations furnish examples of passive pressure; when a heavy load causes the adjacent soil to heave, the passive resistance of the soil has been exceeded.

Determination of the Lateral Pressure.—Three factors are necessary for the complete determination of the lateral pressure, namely, magnitude, direction, and point of application. The theories of lateral pressure cannot be based on the laws of solid bodies, nor on the laws of fluids. A granular material is neither a solid nor a liquid; it is usually a combination of the two. The laws of granular materials have not been fully developed. All theories include some common assumptions:

1.—That the mass is homogenous.

2.—That the material consists of grains, which have the resistance to rolling over each other, called friction.

3.—That the laws of friction hold true.

4.—That the laws of cohesion hold true. Cohesion is usually disregarded, as it cannot be relied on to act at all times.

Wedge of Rupture.—If a retaining wall were removed, some of the retained fill would immediately slip down. Not all the material above the plane of repose would fall at the same time. The first slip is called the wedge or prism of rupture. The surface that is left is fairly plane and is usually called the plane of rupture. Weathering and shock will soon cause the material to assume its natural slope. It is true that clay banks with overhanging tops, ditches with vertical sides, etc., are often seen, but they are merely examples in which the cohesion is acting. Such banks and walls of ditches will soon slip and assume sloping sides unless retained.

Coulomb and Wedge Theories.—The early wedge theories, derived by Coulomb, Mayniel, and Prony assume that the pressure behind a wall is caused by the wedge of rupture exerting a normal pressure on the wall. The resultant was assumed to act at the one-third point of the height of the wall. In 1840, Poncelet developed the general wedge theory, in which the resultant pressure was assumed to act at the angle of friction between the fill and the wall from the normal to the wall. In all the wedge theories, an expression for the lateral pressure is derived in terms of the wedge, and a maximum value obtained by setting the first derivative of the expression equal to zero. Coulomb, in 1773, assumed the width of the wedge as the variable with respect to which the differentiation is performed; all the other theories assume the wedge angle, that is, the angle between the wall and the plane of rupture, as the variable.

Rankine and Stress Theories.—By assuming the mass indefinite in extent and incompressible, the theory of the elasticity of materials can be applied to a unit parallelopiped of volume in the retained mass of earth. In this manner, Rankine obtains his theory. Levy and Boussinesq generalized the stress theory by taking into account the effect of the wall surface, calling it a restraint that causes a discontinuity in the mass.

Many minor theories have been issued, but lack of space prevents even an outline of them in this paper.

Design of the Apparatus

The design of the large-scale apparatus used in the experiments was based on that of Mueller-Breslau in 1906, in that the same types of wall and bin were used. The readings, however, were to be obtained by means of scales. The plans were submitted for correction and criticism to a number of well known engineers. The design required a steel bin 5 ft. high, with side-walls that could move parallel to the test wall, thereby attempting to eliminate the side-wall effect. Professor J. H. Smith, of the University of Pittsburgh, objected to this part of the design. Careful analysis by Professor Braune and the writer disclosed the error in this method of trying to eliminate the effect of the side-walls because any motion of the side-walls, caused by forces existing in the fill, changes the volume and shape of the fill and prevents accurate determination of the existing pressures. William Cain, M. Am. Soc. C. E., warned against too weak a test wall, as deflections of the wall will cause a dissipation of the forces in the fill. The original design called for a thin steel wall, which was to be counterbalanced vertically. D. B. Steinman, M. Am. Soc.

C. E., suggested that the test wall be left entirely free, as a retaining wall is not counterbalanced, but rests on the foundation. He also suggested that the effect of different methods of placing the fill behind the wall be determined. Allen Hazen, M. Am. Soc. C. E., suggested a careful study of the effect of vibration and shock, as well as the question of packing and water content. Robert A. Cummings, M. Am. Soc. C. E., offered the aid of the Special Committee on the Bearing Value of Soils for Foundations, etc., in the proposed investigation, and suggested co-operation to prevent unnecessary duplication of work. A. T. Goldbeck, Assoc. M. Am. Soc. C. E., showed the writer the methods used by the U. S. Bureau of Public Roads in conducting large-scale tests in highway research, and gave some valuable suggestions for the actual testing.

The apparatus, as constructed, has been entirely satisfactory and has proved to be accurate, quite easy to manipulate, and sufficiently flexible, so that changes for the various tests could easily be made.

The following points were considered in the final design as necessary for the accurate determination of the lateral pressures of granular materials. (Fig. 1.) The pressures are measured on a "free" wall that closes one side of the bin. This test wall is supported by two vertical and three horizontal contacts for measuring the horizontal and vertical forces. Provision is made for experiments with vertical and sloped walls, and for walls having various surfaces. The size of the apparatus is such that the arching action of the filling material may be neglected, the values obtained are not miniature, and yet no excessive labor is required. In order to eliminate side-wall resistance, that part of the fill that may tend to move, is made to rest against a steel plate supported on roller wheels bearing against the sides of the bin. The bin is sufficiently large to allow a full plane of repose, has provision for sloping surfaces, and is placed so that the minimum labor is required to fill and empty it. It is well drained and rests on a solid foundation, and is situated so as to be free from vibrations and external shocks.

Description and Construction

Dimensions.—The bin is constructed of reinforced concrete and is 5 ft. 6 in. wide, 9 ft. long, and 6 ft. high in front and 12 ft. high in the back, with walls 6 in. thick. The "free" test wall occupies the entire front of the bin. The back of the bin has an opening 3 ft. wide, which is closed by planks to any required height and which facilitates the emptying of the bin. The top of the bin has a slope of 34°, approximately the natural repose of sand. The length of the bin is sufficient to allow a full plane of repose inside it when it is filled. The side-walls are tied together front and back by a reinforced concrete beam and are supported between a longitudinal beam at the top and the floor at the bottom. The floor of the bin is placed 2½ ft. above the foundation to provide ample room for testing apparatus and also to simplify the operation of filling and emptying. The foundation consists of a 12-in. reinforced concrete slab placed on packed cinders which assure drainage and eliminate vibrations. The floor is 6 in. thick, and is supported on three longitudinal walls, one on each side and one on the center line, which are stiffened by three

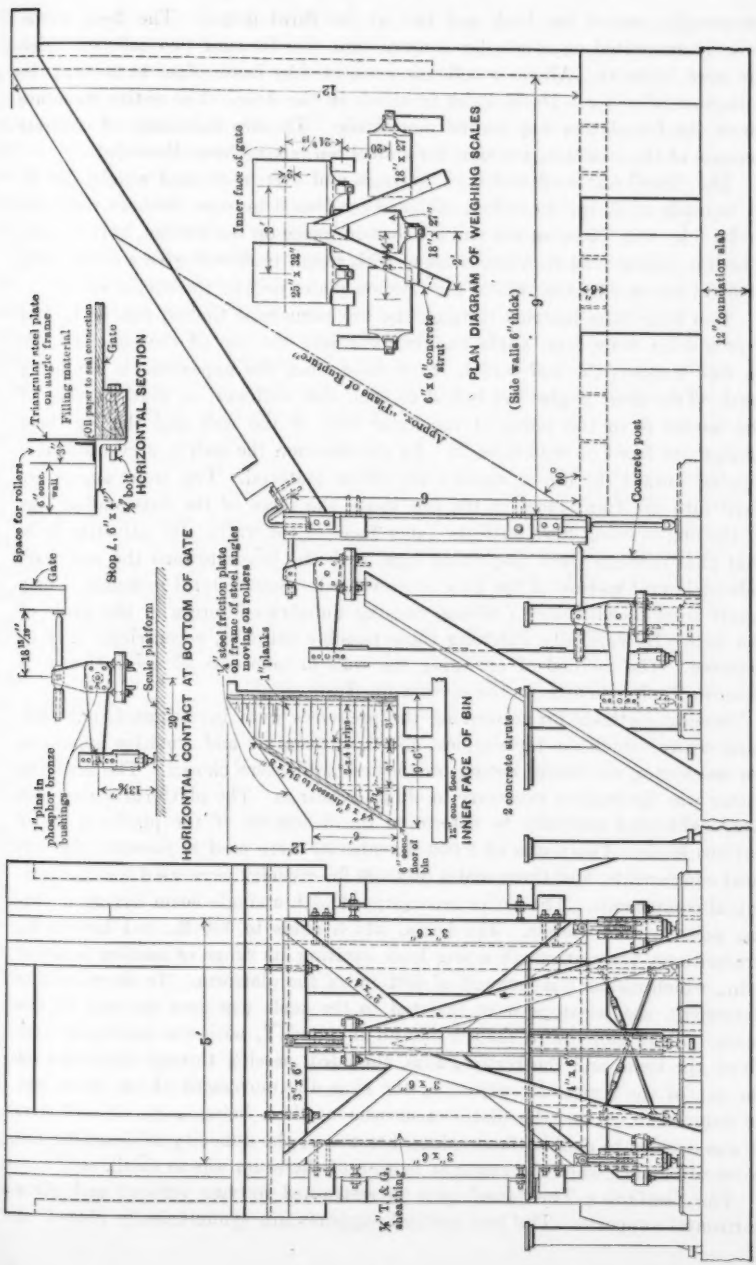


FIG. 1.—EARTH-PRESSURE APPARATUS.

cross-walls, one at the back and two at the third points. The floor, therefore, is supported on a cellular construction, the forward two cells of which are open in front. All these cells are connected by drain pipes to prevent the collection of water. Drain holes were left in the floor. The entire structure above the foundation was poured monolithic. The bin was made of concrete because of the economy, a design for a steel bin having been discarded.

The "free" test wall is 6 ft. 4 in. high and 5 ft. wide, and weighs 290 lb. It is made of $7\frac{7}{8}$-in. sheathing, stiffened by four 3 by 6-in. timbers vertically and a 3 by 6-in. piece on top and a 4 by 6-in. piece on the bottom, horizontally. The bin, which is $5\frac{1}{2}$ ft. wide, is faced with rough timber to give a clear width of 5 ft., except in front, where the friction plates occupy the excess space.

Two bent bolts inserted through the top horizontal timber, can be hooked over a 6 by 6 by $\frac{3}{8}$-in. angle rigidly fixed into the top of the concrete bin. In this manner, the test wall is safely held when the apparatus is not being used. Two short angles are bolted to each side stiffener, so that the face of the angles is in the plane of the inner face of the wall and the legs bear against the front of the bin walls. In this manner, the wall is prevented from moving toward the bin or against the filling material. Two short angles are fixed into the front faces of the bin walls, the faces of the outstanding legs of the angles being flush with the inner faces of the walls. By adjusting bolts that pass through these projecting legs, with the heads toward the test wall, side or lateral motion of the gate is prevented. Such lateral movements may result from eccentricity of filling, causing unbalanced forces in the plane of the wall. By carefully watching these possible contacts, eccentricity may be detected. The method of changing the wall to take care of special cases is described subsequently in the section on Tests.

Several methods for measuring the pressures were investigated, but platform scales, which are the simplest and most accurate and sensitive apparatus for measuring the forces acting on the test wall, were chosen. The deciding factor was the motion necessary to obtain readings. The platform scales used were calibrated carefully to determine the deflection of the platform under various loads. Two scales of 2 000 lb. capacity were used to measure the vertical components, and three scales of 1 000 lb. capacity were used for the horizontal components. The scales are equipped with a single beam having a sliding poise with set-screw. The beam, which reads to 100 lb. and has 0.5-lb. graduations, passes through a trig loop, limiting its range of motion to about 1 in., which means a movement of 0.01 in. of the platform. To decrease this movement, special stops were inserted in the scale cap over the end of the beam. These stops were shaped like an inverted **T**, with the horizontal arm above the beam and the vertical arm, threaded, passing through lock-nuts on the cap of the scale. By adjusting the stop, the movement of the beam can be reduced to $\frac{1}{40}$ in., the platform movement then being $\frac{1}{4\,000}$ in. Later, it was decided to double these values, because of the difficulty of detecting the point of balance when the range of motion of the beam was so small.

The Contacts.—The "free" gate is restrained by two vertical and three horizontal supports. The two vertical supports are symmetrically placed, at

about the quarter-points, and rest directly on the platform scales. (See Fig. 1.) Each vertical support consists of a 1-in. steel rod, the upper end of which is pointed and bears into a steel bearing-plate screwed to the wooden wall. The lower end, which is threaded, is screwed into a steel contact point which rests on a steel plate placed on the scale platform. These contact points allow a certain amount of adjustable motion of the test wall vertically. By raising one or the other, the axis of the wall may be tipped in either direction. In this manner, the wall may be adjusted for eccentricity. When the wall was vertical, the two scales had the same initial or no load reading.

Three horizontal contacts were used to give a stable system of support. As the stress on the wall is a function of the depth, the three supports are placed in a triangle, one at the top in the center, and one in each of the lower corners. Steel plates screwed to the wall at these points formed bearing plates. Each horizontal support consisted of a strut pivoted to a bell-crank, by which arrangement the horizontal pressure was transmitted vertically to the scale platform. The bell-crank arms are in the ratio of 1 to 2, thereby decreasing the reading to one-half the actual pressure. The vertical strut consists of two 5-in. channels, at the lower end of which is an adjustable contact point, consisting of a threaded 1-in. rod in a cast shoe. All parts are made very rigid to reduce deformations to negligible values and all pivots are fitted in phosphor-bronze bushings to reduce frictional resistance. By adjusting the threaded contact points, the test wall may be moved or tipped about its lower edge, and adjusted for any deviation of the plane of the wall from the vertical. The bell-cranks rest on steel plates fixed into stiffened reinforced concrete posts.

The scales were placed so as to make simultaneous readings possible. This required the compact arrangement shown in Figs. 1 and 2. In order to prevent any possible wobbling of the vertical supports, two lateral cross-braces in which turnbuckles were inserted, were later hooked to the rods. These turnbuckles were found to be very useful in adjusting either of the vertical contacts for deviation from the vertical.

The retarding effect of the side-walls has always introduced an element of uncertainty in earth-pressure measurements made in boxes and bins. Winkler* tried to determine the resistance of each wall by duplicating his tests with a center partition. The second set of tests was affected by four wall friction losses, whereas the first set was affected by two friction losses. This method has been shown to be inaccurate. In the apparatus, it was decided to eliminate the resistance effect of the side-walls. The first method decided on, mentioned previously, was to have these walls on rollers, permitting slight motion in a direction parallel to the wall. It was thought that this flexibility would remove the effect of forces along the sides of the fill. The objection that any such motion would cause a change in the volume and shape of the fill, as well as alter the stresses acting therein, caused the abandonment of this design. Professor Braune then suggested a method approximating actual conditions. Behind a retaining wall, a certain part of the fill tends to move toward the wall. The earth back of any given length of wall is not retarded

* Winkler, E., in *Der Civilingenieur*, 1865, pp. 1–11.

by the earth on each side, because that too tends to move. It was decided, therefore, to have part of the side-walls of the bin movable in the direction in which the fill tends to move, that is, in a plane normal to the test wall.

The part of the fill that tends to move is approximately a triangular prism, with bases formed by the surface of the fill, the back of the wall, and the "plane of rupture". The wedge angle for sand is about 30 degrees. Two triangular steel plates of the required shape were made, each of which rests on three phosphor-bronze bushed rollers bearing against the side-walls of the bin. At the points of contact, the concrete was finished very smooth. That part of the side-walls not covered by these plates is faced with timber, held to the sides by bolts through the walls, so that the bin has a uniform width of 5 ft. The steel plates are accurately counterbalanced vertically. Preliminary tests made before the plates were counterbalanced showed the necessity of eliminating the pressure due to the tendency of these plates to slip downward. Two rollers are placed at third points along the plane of rupture to eliminate the friction between the plates and the wooden facing of the walls.

In order to prevent sand flowing into the cracks, all joints were made as close as possible, thin strips of oiled paper, bent into angles, being placed along all the edges of the test wall. The joint between the steel plates and the timber was also covered with oiled paper. The area covered by this paper was so small in comparison with the total exposed area that no account of its effect was taken.

The width of fill being 5 ft., losses due to the frictional resistances of the rollers back of the friction plates may be disregarded. No attempt was made to alter the shape of the plates, as the wedge angle probably averaged very nearly 30 degrees. Some of the tests were made with the plates overbalanced, that is, practically fixed. Although the resulting pressures are probably less, the decrease is not very large. The error thus incurred does not exceed the errors due to non-homogeneous fill or to slight differences or speed of filling.

A brief description of the apparatus has been published,* but on account of the remarkable sensitivity and accuracy of the apparatus, this detailed description is given for the benefit of those who may plan to conduct experiments in the future. An exhaustive description of the design and method of operation is included in a thesis presented by the writer in 1921, and is in the Library of the Graduate School of the University of Cincinnati.

Method of Operation

The most important factor in the operation was to guard against obtaining passive instead of active pressures. Before each test, every contact point was loosened until the test wall was free from the bin. The axis of the wall was placed in a vertical position by adjusting the vertical supports until the two scales showed the vertical components to be equal. A plumb-bob suspended on the wall showed whether the plane of the wall was vertical. All the horizontal contacts were then tightened until the increase in readings on the scales showed that the wall was bearing against the bin. The contacts were then adjusted so as to give the minimum readings on the scales, with the wall as close to the bin

* *Engineering News-Record*, August 25, 1921.

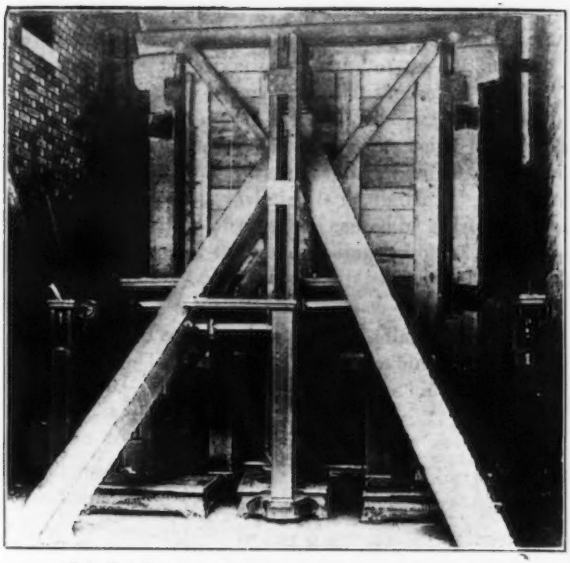


FIG. 2.—TESTING APPARATUS WITH SCALES IN PLACE.

as possible. In this manner, a minimum of opening between the edges of the wall and the bin was obtained. The apparatus was then left for at least 1 hour, usually over night, and the zero readings were taken again. Readings could be taken to ¼ lb., and were so taken at the beginning and end of each set; all other readings were taken to ½ lb.

Before any material was placed in the bin, the riders on the scale-beams were moved out, that is, the scales were overbalanced, to a point beyond the next reading. After a definite height of fill had been placed, the riders were slowly moved back, and the readings were taken as the beam just rose from the lower support of the trig loop. As soon as the reading was taken, the rider was run out again to a point beyond the next possible reading. This method probably reduced the motion of the gate to one-half its possible movement, or to $\frac{1}{8\,000}$ in. vertical and $\frac{1}{16\,000}$ in., horizontal, which are values less than the elastic deformation of the wall, or of any retaining wall. The pressure recorded, therefore, is the active pressure of the fill, and not the passive resistance of the fill caused by the movement of the wall.

To test the sensitiveness of the method, readings were often repeated in as short an interval of time as possible, the results being usually identical and seldom varying by ½ lb. As the average readings were about 500 lb., and sometimes as much as 1 500 lb., this variation is negligible.

To determine the variation of pressures caused by greater movements of the scale-beams, and, therefore, of the wall, readings of one fill were taken for the scale-beams, and, therefore, of the wall, readings of one fill were taken for the all the other tables given, the scales are numbered, as follows:

No. 1.—The upper horizontal reading.
No. 2.—The lower left (facing the bin) reading.
No. 3.—The lower right (facing the bin) reading.
No. 4.—The left vertical reading.
No. 5.—The right vertical reading.

The vertical readings less the zero readings give the actual vertical pressures, whereas the horizontal readings must be doubled.

TABLE 1.

	Scale Readings.				
	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.
Scale-beam in restricted position	554	464	528	985	701
Scale-beam at highest point of trig loop	533	451	510	951	675
Scale-beam at middle point of trig loop	541	459	518	965	683
Scale-beam at lowest point of trig loop (same position as when restricted)	558	465	527	985	701

The total movement of the beams was ½ in., corresponding to a movement of the wall of $\frac{1}{2\,000}$ in. vertical and $\frac{1}{4\,000}$ in. horizontal, which movements had no effect on the recorded pressures. Similar results were obtained in the other two tests to determine the effect of the movements of the wall. It may be concluded, therefore, that, for such values, the pressures are unaffected. The

effect of larger movements could not be determined, because the trig loops are not removable.

In addition to the readings of the scales, a record was kept of the temperature, time of day, state of the weather, and age of the fill in the bin. In the first few tests, readings were taken for every 3-in. increase in height of fill. This was changed to 6-in. increments, as too much time was required, both during the test and in computing results.

The five readings give the magnitude, direction, and point of application of the resultant earth pressure. The vertical component is the sum of the readings on Scales Nos. 4 and 5, less the initial readings on these two scales. The total horizontal component is twice the sum of the readings on Scales Nos. 1, 2, and 3, after the initial readings on these scales have been subtracted. The magnitude of the resultant can then be found. The direction is the angle the tangent of which is the ratio of the total vertical to the total horizontal component. By taking moments about the intersection of a plane through the vertical supports of a plane through the lower horizontal supports, the effect of these four forces is eliminated and the equality of moments caused by the upper horizontal support and by the total resultant is obtained. The upper horizontal support is 6.04 ft. above this line of intersection. Resolving the resultant into a vertical component acting along the back of the wall and a horizontal component acting at a distance, x, above this axis of moments, and equating moments, the height, x, or the resultant pressure is obtained. A vertical wall has no moment, because the vertical supports are placed under its center of gravity. When additional parts were attached to the wall, for glass-backed walls, or oblique walls, the moment of additional parts was considered.

Tests were performed with glass and sheet metal directly fixed to the back of the wooden wall. For walls sloping toward the fill, a "false" wall was attached to the wall at three points. At the foot, it was spiked to the main wall and at the mid-point and at the top, the two walls were connected by heavy timber struts. Similar connections were used for the wall sloping away from the fill. Because of the large vertical components in the latter case, sixteen wires were also run from the lowest point of the overhanging wall to various points in the main wall to aid in transmitting the vertical component and in decreasing the deflection of the "false" wall. In both these cases, the additional parts were inside the bin, and care was taken to prevent contact on the floor or binding along the sides. The results for walls with considerable slope are not as accurate as those for vertical walls. Some of these tests were repeated to obtain check values.

A sample of the material used in each test was taken. Air-tight cans, holding about 15 lb. of material, were filled when each test was about half finished. The density, natural slope, coefficient of internal friction, coefficient of internal resistance, and moisture content of this material were later carefully determined. The coefficients of friction and resistance (kinetic and static friction) of the material of the fill on itself and on the back of the wall were also determined. The coefficient of internal resistance was measured by determining the force required to start in motion a weighed quantity of material

resting on the same material. The force required to keep the mass in uniform motion gave the coefficient of internal friction. The material was placed in a cubical bottomless box that held 5½ lb. of sand, and the lower edges were beveled so that, practically, only the sand was in contact. The coefficient of internal resistance was taken as a measure of the combined effect of friction and cohesion; whereas the coefficient of internal friction was due to friction alone, as there can be no cohesion during motion. The angles corresponding to these coefficients are called the angles of internal friction and internal resistance, respectively. The angle of internal resistance in all cases (for sand) was found to be about 90% of that of natural repose.

Outline of Tests

The following is an outline of the tests performed:

I.—Variation in physical properties of the sand under various conditions.
II.—Lateral pressure of horizontal fills against a vertical wooden wall.
III.—Lateral pressure of horizontal fills against a glass-backed vertical wall.
IV.—Lateral pressure of horizontal fills against a sheet-metal-backed vertical wall.
V.—Lateral pressure of sloping fills against a vertical wooden wall.
VI.—Lateral pressure of sloping fills against a glass-backed vertical wall.
VII.—Lateral pressure of sloping fills against a sheet-metal-backed vertical wall.
VIII.—Lateral pressure of horizontal fills against walls with positive back batter.
IX.—Lateral pressure of horizontal fills against walls with negative back batter.
X.—Lateral pressure of sloping fills against a wall with a positive back batter of 1:4.
XI.—Lateral pressure of sloping fills against a wall with a negative back batter of 1:6.
XII.—Lateral pressure of irregular fills against a vertical wooden wall.
XIII.—Effect of settling and changes of temperature on the lateral pressure of horizontal fills against a vertical wall.
XIV.—Effect of static loads on the lateral pressure of horizontal fills against a vertical wall.
XV.—Effect of moving loads on the lateral pressure of tamped horizontal fills against a vertical wall.
XVI.—Effect of static loads on the lateral pressure of horizontal fills against a wall with positive back batter of 1:4.

A few of these tests were made during the spring of 1921 and the remainder during the fall and winter of 1921-22. The filling and emptying of the bin eighteen times required the shoveling of about 150 tons of sand.

I.—Physical Properties of the Sand.

River sand excavated from the subway cut on Canal Street, Cincinnati, Ohio, was used. When first obtained the specific gravity of the sand was about

2.6 and the moisture content was 9 per cent. Shoveled and slightly tamped into a box, it weighed 100 lb. per cu. ft. Exposure caused a slow drying and, during the last tests, the moisture content was as low as 3 per cent. The first tests, therefore, were made with a "damp" sand, and the remainder with a "humid" sand, in accordance with the definitions of the Special Committee of the Society to Codify Present Practice on the Bearing Value of Soils for Foundation, etc. Moisture had considerable influence on the other physical properties. A mechanical analysis is as follows:

Sieve number	4	8	10	14	28	48	100
Percentage passed	92	85	81	78	63	10	2½

The material contained very little clay and dirt. The natural slope was determined by pouring the sand on a table to form a cone about 12 in. high, the slope being measured along six elements. Four determinations of coefficients of friction and of resistance were made for every sample. At first, the coefficient of resistance was measured by using a spring scale and also by weights over a fixed pulley. The results were practically identical, so that only the spring-scale method was used in the later tests. A complete set of physical tests was also made on dry sand.

TABLE 2.

	Location of Material.		
	Surface.	6-ft. depth.	10-ft. depth.
Weight per cubic foot	100 lb.	98 lb.	101 lb.
Moisture content	9%	9%	9%
Angle of natural slope	40° 0′	40° 40′	43° 40′
Angle of internal friction	34° 0′	34° 30′	35° 30′

The unit weight of the material was a minimum (90 lb. per cu. ft.) when the moisture content was between 4 and 5 per cent. As the moisture content of the sand was increased, the natural slope was increased likewise, dry sand having a slope of 30° 15′ and sand with 9% moisture a slope of about 40 degrees. The same holds true for the angle of internal friction, which was 28° 15′ for dry sand and 34° for the very moist sand; and also for the angle of internal resistance, which was 28° 15′ for dry sand and 37° for the very moist sand. It should be noted that, for the dry sand, there is no difference between the angles of friction and resistance. This difference was 2° for a moisture content of 3% and 3° for a moisture content of 9 per cent. The intermediate values are fairly consistent. The difference between the internal friction and internal resistance is caused by the adhesion due to a water film around each particle of sand. This property was called "cohesion" by Coulomb, and the name is still used, although it is difficult to understand how cohesive forces can act between particles or grains of a fill separated by a film of water. This film is always present, as there are no perfectly dry materials back of retaining walls.

In order to determine whether the physical properties of the material varied with the height of the fill, two loosely built wooden boxes, with no covers, were

embedded in the sand for three months, from June to September, one cube being 6 ft., and the other 10 ft., below the surface.

Physical tests on the material in these cubes and of the surface material are given in Table 2.

Because of the small samples used, little can be concluded from these tests, and further investigation is necessary.

II.—Lateral Pressure of Horizontal Fills Against a Vertical Wooden Wall.

Several tests were made during April and May, 1921, while the apparatus was being adjusted and calibrated. The following data were obtained after it was found that results could be closely duplicated and the apparatus was completed and satisfactory. In Test No. 1 (Table 3), all figures and computations are given in order to show the method. The columns contain the following:

Column 1.—Height of fill in bin.

Columns 2, 3, 4, 5, and 6.—The actual readings on Scales Nos. 1, 2, 3, 4, and 5.

Columns 7, 8, 9, 10, and 11.—The pressure recorded, obtained by subtracting the initial readings.

Column 12.—The top horizontal pressure, H_1, double the reading.

Columns 13 and *14.*—The bottom horizontal pressures, H_2 and H_3, double the readings.

Column 15.—The total horizontal component, the sum of H_1, H_2, and H_3.

Column 16.—The total vertical component, the sum of the vertical scale readings, found in Columns 10 and 11.

Column 17.—The total pressure, the square root of the sum of the squares of the two total components.

Columns 18, 19, and *20.*—The horizontal component, the vertical component, and the total pressure per foot length of wall.

Column 21.—The ratio of the total vertical to the total horizontal component.

Column 22.—The angle the tangent of which is the ratio of the total vertical to the total horizontal component. Columns 23 to 27 furnish data on the point of application of the total pressure, by the formula developed for each type of test wall. The results are compared with three theories. The Coulomb theory, in which the wall friction is disregarded, is shown by the coefficient of $(\frac{1}{2} y h^2)$ as N. In the case of a vertical wall, it is equal to the horizontal component and takes no account of a vertical component. In the cases of inclined walls, the horizontal component, as given by this theory, is denoted by N_H. The general wedge theory, in which the friction on the wall is taken into account, is shown by the coefficient of $(\frac{1}{2} y h^2)$ as C. The horizontal component is H. The Rankine theory is shown by the coefficient of $(\frac{1}{2} y h^2)$ as R. The horizontal component is K. It was decided to compare the theories by the coefficients of the quantities $(\frac{1}{2} y h^2)$, in order to compare the results with hydrostatic pressures, y being the density of, and h the height of, the fill.

TABLE 3.—TEST NO. 1.

May 12, 1921, 2-5 P.M.
Weather: Rain.
Temperature: 10-12° cent. (50-55° Fahr.)
Moisture content = 9%.

Physical Properties:
Weight per cubic foot = 100 lb.
Angle of internal resistance = 37°
" " internal friction = 34°
" " natural slope = 40°
" " friction on wall
(static friction) dry = 30°
" " damp = 32° 30'.

Height of fill, h, in bin, in inches.	Readings on Scales, in Pounds.					Scale Measurements, in Pounds.					Horizontal Pressures, in Pounds.			Total horizontal component, in pounds.
	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	H_1	H_2	H_3	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
0	95	60.5	60	150.5	157	.....	.....	.....	.....	.....	.....	.....	.....	.....
6	96	62	64	152.5	160	1	1.5	4	2	3	2	3	8	13
9	97	66	70	159	165	2	5.5	10	8.5	8	4	11	20	35
12	97.5	78	82	178	183	2.5	17.5	22	27.5	26	5	35	44	84
15	99	90	95.5	199.5	194	4	29.5	35.5	49	37	8	59	71	138
18	100	100	104	214	205	5	39.5	44	63.5	48	10	79	88	167
21	102	112	117	234.5	222	7	51.5	57	84	65	14	103	114	231
24	106	124.5	132.5	259	247	11	60	72.5	108.5	90	22	120	145	287
27	113	137	146	284	269	18	76.5	86	133.5	112	36	153	172	361
30	122	146.5	163	296.5	308	27	86	108	146	145	54	172	206	432
33	138	171.5	188	345	333	43	111	128	194.5	176	86	222	256	564
36	148	194	212	377	372	53	133.5	152	226.5	215	106	267	304	657
39	161	200.5	231	403	412	66	140	171	252.5	255	132	280	342	754
42	178	215	248	422.5	432	83	154.5	188	272	274	166	309	376	851
45	187	232.5	268	465	482	92	172	208	314.5	325	184	344	416	944
48	209	246	287	494.5	522	104	185.5	227	344	365	208	371	454	1 033
51	229	263	305	526.5	560.5	134	202.5	245	376	403.5	268	405	490	1 163
54	250.5	278	326.5	561	588	155.5	217.5	266.5	410.5	431	311	435	533	1 279
57	274.5	295	349.5	610.5	632	179.5	234.5	289.5	460	475	359	469	579	1 407
60	303	313	371	657	676	208	252.5	311	506.5	519	416	505	622	1 543
63	332	326.5	385	699	721	237	266	325	548.5	564	474	532	650	1 656
66	386	362	439	791	785	291	301.5	379	640.5	628	582	603	758	1 973
69	442	379	459	845.5	842.5	347	318.5	299	695	685.5	694	637	798	2 129
72	471	397	487	907	896	376	336.5	427	756.5	739	752	678	854	2 273

Column No. 1 = top horizontal contact scale reading.
Columns Nos. 2 and 3 = bottom horizontal contact scale reading.
Columns Nos. 4 and 5 = vertical contact scale reading.

TABLE 3.—(*Continued*).

Total vertical component, in pounds	Total pressure, in pounds.	Pressures per Foot, in Pounds.			$\tan \phi' = \frac{V}{H}$	Inclination of P	Point of Application.			$x = \frac{\text{Column 25}}{H}$	$\frac{x}{h}$
		H	V	P		ϕ'	$6.04H_1$.	$0.23V$	Sum of Columns 23 and 24.		
(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)
.....	....	....	....	....	.....	.....	....	....	....	....	.....
5	14	2.6	1.0	2.8	0.385	21° 00′	12	1	13	1.0	.....
16.5	39	7.0	3.5	7.5	0.472	25° 15′	24	4	28	0.80	.....
53.5	100	16.8	10.7	19.9	0.637	32° 30′	30	12	42	0.50	.....
86	163	27.6	17.2	30.5	0.623	31° 55′	48	20	68	0.44	0.35
111.5	198	38.4	22.3	39.9	0.667	33° 40′	60	26	86	0.52	0.344
149	275	46.2	29.8	55.0	0.645	32° 50′	84	34	118	0.51	0.292
198.5	349	57.4	39.7	69.8	0.692	34° 40′	133	46	179	0.625	0.312
245.5	437	72.2	49.1	87.3	0.680	34° 10′	217	56	273	0.757	0.336
291	521	86.4	58.2	104.2	0.674	34° 00′	326	67	393	0.910	0.364
370.5	675	112.8	74.1	134.9	0.658	33° 20′	519	85	604	1.07	0.389
441.5	792	131.4	88.3	158.3	0.673	33° 55′	640	102	742	1.13	0.378
507.5	909	150.8	101.5	181.8	0.673	33° 55′	797	117	914	1.21	0.373
546.5	1011	170.2	109.2	202.2	0.642	32° 40′	1001	126	1127	1.33	0.380
639.5	1140	188.8	127.9	225.0	0.678	34° 10′	1111	147	1258	1.33	0.355
709	1253	206.6	141.8	250.6	0.685	34° 25′	1258	163	1421	1.38	0.345
779.5	1400	232.6	155.9	280.0	0.670	33° 50′	1620	179	1799	1.55	0.365
841.5	1531	255.8	168.3	306.2	0.659	33° 20′	1880	193	2073	1.62	0.360
935	1690	281.4	187.0	338.0	0.665	33° 40′	2152	215	2367	1.68	0.354
1025.5	1853	308.6	205.1	370.6	0.665	33° 40′	2515	236	2751	1.78	0.356
1112.5	1995	331.2	222.5	399.0	0.673	33° 55′	2862	256	3118	1.88	0.358
1268.5	2346	394.6	253.7	469.2	0.642	32° 40′	3508	292	3800	1.93	0.351
1370.5	2532	425.8	274.1	506.4	0.644	32° 50′	4190	315	4505	2.12	0.369
1495.5	2721	454.6	299.1	544.2	0.659	33° 20′	4540	344	4884	2.15	0.359

TABLE 4.—TEST No. 2.

May 4, 1921 2-3 P.M., 0-3 ft.
May 5, 1921, 2-4 P.M., 3-6 ft.
Weather: Clear.
Temperature: 60-65° Fahr.

Physical Properties:
Same as in Test No. 1 (Table 3).

Height of fill, h, in inches.	Total Pressures, in Pounds.				Pressures per Foot, in Pounds.			Inclination of P	$\frac{x}{h}$ Ratio of height of P : h
	H_1.	H.	V.	P.	H.	V.	P.	ϕ'.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
6	20	66	4	66	13.2	0.8	13.2	3°	.....
9	22	78	9	78	15.6	1.8	15.6	6°	.....
12	24	100	23	103	20.0	4.6	20.6	13°	.....
15	26	128	47	136	25.6	9.4	26.3	20°	.....
18	30	170	80	188	34.0	16.0	37.6	25°	.....
21	32	206	111	234	41.2	22.2	46.8	23° 40′	.....
24	38	268	160	312	53.6	32.0	62.4	30° 50′	.....
27	42	334	204	391	66.8	40.8	78.2	31° 30′	.....
30	50	386	253	462	77.2	50.6	92.4	33° 10′	.....
33	60	476	300	563	95.2	60.0	112.2	32° 20′	.....
36	115	650	387	757	130.0	77.3	151.4	29° 55′	.....
39	154	762	439	880	152.4	87.8	176.0	30° 00′	.....
42	166	798	486	934	159.6	97.2	186.8	31° 20′	.....
45	190	878	555	1 040	175.6	111.0	208.0	32° 20′	0.398
48	244	1 026	654	1 215	205.0	130.8	242.3	32° 50′	0.404
51	318	1 222	768	1 443	244.4	153.6	288.6	34° 10′	0.395
54	352	1 302	823	1 541	260.4	164.6	308.2	32° 10′	0.389
57	396	1 400	893	1 661	280.0	176.6	332.2	32° 30′	0.391
60	469	1 562	992	1 851	312.4	198.3	370.1	32° 20′	0.391
63	556	1 706	1 071	2 016	341.2	214.2	403.2	32° 10′	0.402
66	700	1 986	1 224	2 333	397.2	244.8	466.6	31° 40′	0.404
69	752	2 112	1 344	2 502	422.4	268.8	500.4	32° 30′	0.400
72	834	2 292	1 443	2 708	458.4	288.6	541.6	32° 10′	0.390

To find the height of the total pressure, note that the intersection of the plane through the vertical contacts and through the lower horizontal contacts is level with the bottom of the fill. The upper horizontal contact is 6.04 ft. above this axis; the back of the wall (along which the vertical component, V, acts), is 0.23 ft. from the axis. Calling x the height of the horizontal component,

$$H\,x = 6.04\,H_1 + 0.23\,V,$$

or

$$x = \frac{6.04\,H_1}{H} + \frac{0.23\,V}{H}.$$

The weight of the wall has no effect, because the vertical supports are below the center of gravity. The large number of data in each test makes it impractical to present all the tests in full. Except for the first test, only the essential data are presented.

The bin was filled in two days. The readings for the height of 36 in. are the averages of the readings at the end of the first day and at the beginning of the second. The high values for $\frac{x}{h}$ are due to the settling of the fill over night (see Test XIII for the effect of settling). The low values of ϕ' at the beginning of the test are caused by the dry wall.

The Coulomb and Rankine formulas are:

$$P = \tfrac{1}{2}\,y\,h^2 \tan^2 \tfrac{1}{2}\,(90° - \phi) = N\,(\tfrac{1}{2}\,y\,h^2) = R\,(\tfrac{1}{2}\,y\,h^2).$$

Both theories give a horizontal resultant.

TABLE 5.—Showing Comparison Between Experiment and Theories for the Case of a Horizontal Fill and a Vertical Wall.

(See Fig. 3.)

Height of fill in bin, in inches.	Pressures per Foot of Wall, in Pounds.						$\frac{1}{2} y^* h^2$†	$\frac{H}{\frac{1}{2} y h^2}$	$\frac{P}{\frac{1}{2} y h^2}$
	Test No. 1.		Test No. 2.		Average.				
	H.	*P.*	*H.*	*P.*	*H.*	*P.*			
12	16.8	19.9	20.0	20.6	18.4	20.3	50	0.368	0.406
15	27.6	30.5	25.6	26.3	26.6	28.4	78	0.341	0.369
18	33.4	39.9	31.0	37.6	33.7	38.8	112.5	0.300	0.345
21	46.2	55.0	41.2	46.8	43.6	50.9	153	0.284	0.332
24	57.4	69.8	53.6	62.4	55.5	66.1	200	0.278	0.331
27	72.2	87.3	66.8	78.2	69.5	82.9	253	0.275	0.328
30	86.4	104.2	77.2	92.4	81.7	98.3	312	0.262	0.315
33	112.8	134.9	95.2	112.2	104.0	123.6	378	0.276	0.327
36	131.4	158.3	130.0	151.4	130.7	154.9	450	0.290	0.344
39	150.8	181.8	152.4	176.0	151.6	178.9	529	0.287	0.338
42	170.2	202.2	159.6	186.8	164.9	194.5	611	0.270	0.318
45	188.8	225.0	175.6	208.0	182.2	216.5	702	0.260	0.309
48	206.6	250.6	205.2	242.3	205.9	246.5	800	0.257	0.308
51	232.6	280.0	244.4	288.6	238.5	284.3	902	0.265	0.316
54	253.8	306.2	260.4	308.2	253.1	307.2	1 012	0.250	0.301
57	281.4	338.0	280.0	332.2	280.7	335.1	1 128	0.249	0.297
60	306.6	370.6	312.4	370.1	310.5	370.3	1 250	0.248	0.296
63	331.2	399.0	341.2	403.2	336.2	401.1	1 380	0.244	0.291
66	394.6	469.2	397.2	466.6	395.9	468.9	1 510	0.262	0.310
69	425.8	506.4	422.4	500.4	424.1	503.4	1 650	0.257	0.305
72	451.6	541.2	458.4	541.6	456.5	542.9	1 800	0.254	0.302
Averages from 24 in.								0.264	0.311

* $y = 100$ lb. per cu. ft.

† h = height, in feet.

If we take

$\phi = 40°$, the natural slope, $R = N = \tan^2 25° = 0.218$

$\phi = 37°$, the internal resistance, $N = \tan^2 26\frac{1}{2}° = 0.248$

$\phi = 34°$, the internal friction, $N = \tan^2 28° = 0.283$

The total pressure in the general wedge theory for this case is:

$$\frac{N}{\cos \phi'} = C_1 = \frac{0.248}{\cos 32° 30'} = 0.294;$$

for ϕ = internal resistance; ϕ' = friction along the wall. In C_1, the nature of the wall has been assumed to have no effect on the horizontal component. Assuming that it has, for $\phi = 37°$, $\phi' = 32° 30'$, $n = 0.818$, $C_2 = 0.230$.

$$H = C_2 \cos \phi' = 0.193; \quad n = \sqrt{\frac{\sin (\phi + \phi') \sin \phi}{\cos \phi'}}.$$

Conclusions.—Tests Nos. 1 and 2 give the same results, and agree quite closely. It is evident that a vertical component exists. The average of all the readings gives 33° for the angle of inclination of the resultant pressure. The fill was quite moist, and the wooden wall, when it was exposed after the test, was quite damp. The coefficient of friction between the sand and a wet yellow pine board, across the grain (identical with the case of the test wall), was found to be 32° 30′.

The point of application varies from $0.350\,h$ to $0.400\,h$, depending on conditions. The cause and nature of this variation, as well as that of the inclination, are considered in subsequent tests.

For this case, the Rankine and Coulomb theories give the same formula, which seems to give the horizontal component very closely, if the value given to ϕ is the angle of internal resistance. Low values are obtained by using ϕ as the angle of natural slope. Using ϕ as the angle of internal friction gives high values, which is to be expected, as the effect of cohesion can then be disregarded. The average value for $\frac{H}{\frac{1}{2}\,y\,h} = 0.264$, which corresponds almost exactly to $N = \tan^2 \frac{1}{2}\,(90° - \phi)$, in which $\phi = 36$ degrees. The experimental value of the angle of internal resistance was 37 degrees. The general wedge theory gives low values both for the horizontal component and for the total pressure, but the ratio is the same as that found. The curves shown on Fig. 3 give a graphical comparison between the two experimental results, the Coulomb-Rankine theory for $\phi =$ natural slope; and the formula,

$$H = \tfrac{1}{2}\,y\,h^2 \tan^2 \tfrac{1}{2}\,(90 - \phi_1),$$

in which ϕ_1 is the angle of internal resistance. It was first assumed that ϕ should be the angle of internal friction. Graphical comparison with the experimental curves showed a constant difference between theory and experiment, evidently due to disregarding the effect of cohesion. Introducing the effect of the cohesion into the value of ϕ has solved the difficulty.

III.—Lateral Pressure of Horizontal Fills Against a Glass-Backed Vertical Wall.

Closely fitting sheets of glass were fastened to the back of the wooden wall to a height of 24 in. above a 2-in. strip of wood nailed at the bottom of the wall, and this held the glass in place and prevented slipping. Accuracy could not be expected, because of the small heights used and the method of fastening.

In general, the conclusions from the test with a wooden wall hold also for a smoother wall. The large value of the angle of inclination is due to the method of holding the glass in place, the bottom clamp allowing the fill to rest directly on the wall. It should be noted that ϕ' decreases as the height of the fill increases, and a greater proportion of glass surface is effective. This is well shown in the test of sloped surfaces against a glass wall.

IV.—Lateral Pressure of Horizontal Fills Against a Sheet-Metal-Backed Vertical Wall.

In order to determine whether the nature of the wall had any effect on the lateral pressure, sheet metal with a clean, polished surface was nailed to a height of 3 ft. directly to the wall.

The results check the conclusions given previously. The lateral horizontal pressure is, therefore, independent of the nature of the wall. The high value for the angle of inclination was expected. The surface of the sheet metal was rusted and roughened by the contact and rubbing of the damp sand.

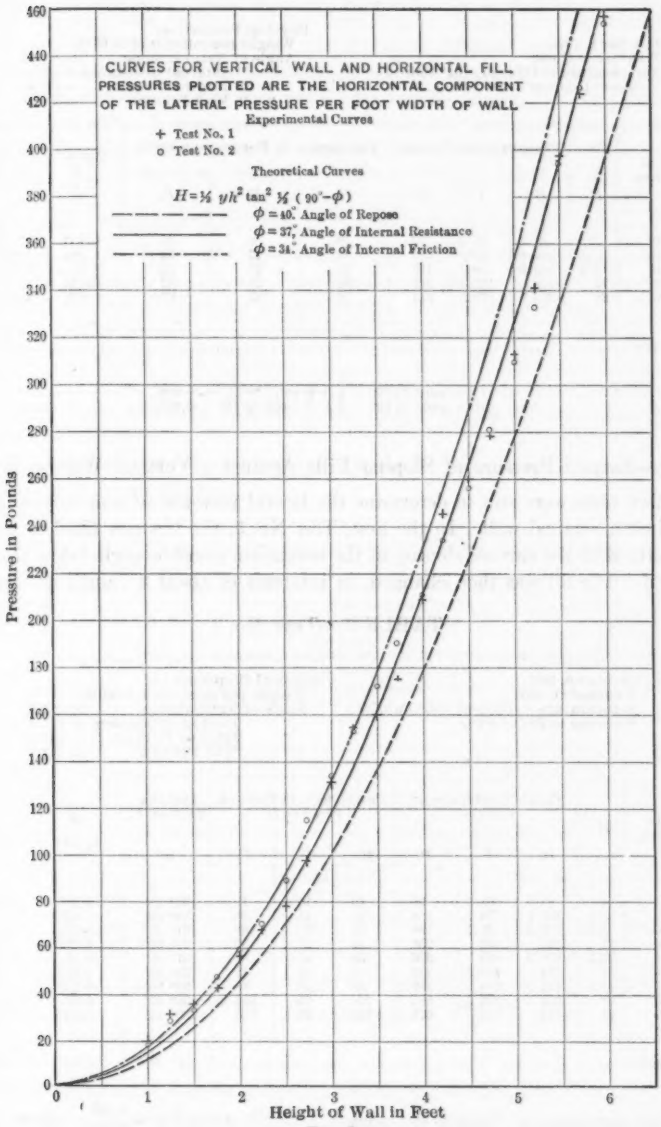

CURVES FOR VERTICAL WALL AND HORIZONTAL FILL
PRESSURES PLOTTED ARE THE HORIZONTAL COMPONENT
OF THE LATERAL PRESSURE PER FOOT WIDTH OF WALL
Experimental Curves
+ Test No. 1
o Test No. 2
Theoretical Curves
$H = \frac{1}{2}\gamma h^2 \tan^2 \frac{1}{2}(90° - \phi)$
$\phi = 40°$ Angle of Repose
$\phi = 37\frac{1}{2}°$ Angle of Internal Resistance
$\phi = 35°$ Angle of Internal Friction
Pressure in Pounds
0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300 320 340 360 380 400 420 440 460
0 1 2 3 4 5 6
Height of Wall in Feet

FIG. 3.

TABLE 6.—Test No. 3.

October 8, 1921.
Weather = Clear.
Temperature = 12° cent. (54° Fahr.)
Moisture Content = 4.9%

Physical Properties:
Weight per cubic foot = 93 lb.
Angle of Natural Slope = 35°30′
" " Internal Resistance = 33°50′
" " Internal Friction = 31°50′
" " Wall Friction = 28°10′

h, in inches.	Total Pressures, in Pounds.				Pressures, in Pounds per Foot.			$\text{Tan } \phi' = \frac{V}{H}$	ϕ'	$\frac{H}{\frac{1}{2} \gamma h^2}$
	H_1.	H.	V.	P.	H.	V.	P.			
12	2.5	61.5	46.5	77	12	9	15	0.755	37°	0.528
15	5.5	107.5	74.5	133	22	15	31	0.694	35°	0.304
18	9.5	144.5	98.5	175	29	20	35	0.681	34°	0.276
21	14	172	122.5	211	34	25	42	0.712	35°	0.240
24	23	263.5	172.5	314	53	35	63	0.655	33°	0.285
Average										0.273

$C_1 = \tan^2 \frac{1}{2} (90 - \phi_1) = \tan^2 28°05′ = 0.284.$
$C = \tan^2 \frac{1}{2} (90 - \phi) = \tan^2 27°15′ = 0.265.$

V.—Lateral Pressure of Sloping Fills Against a Vertical Wooden Wall.

Two tests were run to determine the lateral pressure of non-horizontal fills against a vertical wall. In the first, Test No. 5, the bin was filled to a depth of 5 ft., with the surface sloping at the maximum possible angle below the horizontal. The fill was then changed, in intervals of about 3°, until the greatest

TABLE 7.—Test No. 4.

October 9, 1921.
Weather : Clear.
Temperature = 17° cent. (63° Fahr.)
Moisture in Fill = 4.2%

Physical Properties:
Weight per cubic foot = 94 lb.
Angle of Natural Slope = 38° 30′
" Internal Resistance = 37° 30′
" Internal Friction = 34° 50′
" Wall Friction = 28° 00′

h, in inches.	H_1.	Total Pressures, in Pounds.			Pressures, in Pounds per Foot.			Inclination of P.	$\frac{H}{\frac{1}{2} \gamma h^2}$	$\frac{V}{\frac{1}{2} \gamma h^2}$
		H.	V.	P.	H.	V.	P.	ϕ'		
14	4	81	51.5	96	16	10	19	32° 25′	0.250	0.297
17	5.5	106.5	74.5	128	21	15	26	35° 00′	0.224	0.277
20	8.5	160.5	97.5	188	32	20	38	31° 15′	0.244	0.290
23	13	220	135	258	44	27	52	31° 35′	0.255	0.301
26	20.5	279.5	168	326	56	34	65	31° 00′	0.254	0.295
29	31	354	210.5	412	71	42	82	30° 45′	0.258	0.298
32	41.5	415.5	245.5	483	83	49	95	30° 30′	0.248	0.284
35	58	501	293.5	580	100	59	116	30° 20′	0.251	0.291
38	78	584	340	675	117	68	135	30° 15′	0.248	0.286
Averages								31° 00′	0.248	0.291

$N = R = \tan^2 \frac{1}{2} (90 - \phi) = \tan^2 25° 45′ = 0.232.$

$N_1 = R_1 = \tan^2 \frac{1}{2} (90 - \phi_1) = \tan^2 26° 15′ = 0.243.$

$$C = \frac{N_1}{\cos \phi'} = \frac{0.243}{0.883} = 0.275 \ (\phi' = 28°)$$

$$C = \frac{0.243}{0.857} = 0.284 \ (\phi' = 31°).$$

possible slope above the horizontal was obtained. In this manner, a comparison is had of all possible surfaces with a fixed height of wall. In the second test, No. 6, the surface slope was kept constant and different heights of fill were tested. It was found to be impractical to use the angle of repose as the slope to be tested, because the material would slip and the larger particles would roll down to the wall. A slope as steep as possible was used. Further tests with sloped surfaces and broken surfaces are described subsequently. (See Fig. 4.)

Table 8, which shows the results of Test No. 5, furnishes the following data: The value of $h = 5$ ft.; α, the inclination of the wall, is zero. In Column 1 is given values of the surface inclination, ε, negative below the horizontal and positive above it. As it was difficult to obtain plane surfaces without considerable disturbance of the fill, a rod was fixed at the proper height across the bin, 8 ft. back of the wall, and the sand filled to a plane surface by eye. The surfaces below the horizontal were probably somewhat concave, whereas those above the horizontal were convex.

Column 2 gives the top horizontal pressure. The first two readings were probably a little too high, because of the impact of the fill as it was thrown against the wall.

Columns 3, 4, and 5 give the total horizontal and vertical components and the total pressure on the wall.

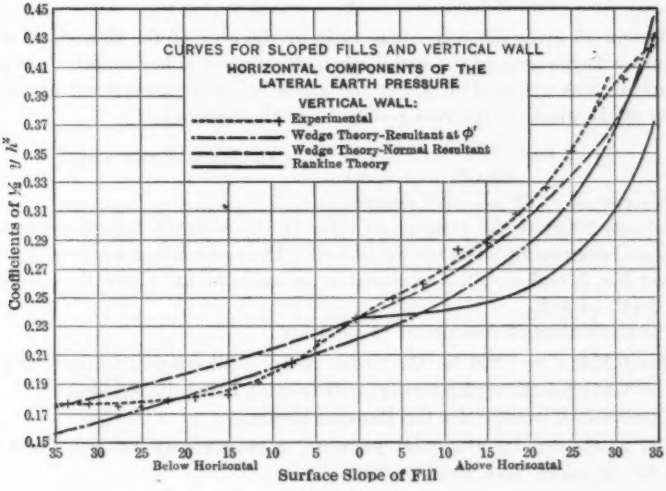


FIG. 4.

Column 6 gives the angle of inclination of the resultant.

Columns 7, 8, and 9 give the horizontal and vertical components and the total pressure per foot of width of the wall.

Column 10 gives the ratio of the height of inclination to the total height of fill. This value is found in the same way as in Tests Nos. 1 and 2.

Column 11 gives the coefficient of $(\frac{1}{2} y h^2)$ for the experimental horizontal component per foot of wall.

Column 12 gives the coefficient, J, of the formula: Horizontal component $= J\ (\frac{1}{2} y h^2)$, in which the general wedge theory is used, disregarding friction on the wall, and taking ϕ as the internal resistance.

$$J = \left(\frac{\cos \phi}{n+1}\right)^2 ; \; n = \sqrt{\frac{\sin \phi \sin (\phi - \varepsilon)}{\cos \varepsilon}}.$$

Column 13 gives the coefficient, K, of the formula: Horizontal component $= K\ (\frac{1}{2} y h^2)$, in which the Rankine theory is used. The theory does not apply for negative slopes. ϕ is taken as the angle of internal resistance.

$$K = \cos^2 \varepsilon \frac{\cos \varepsilon - \sqrt{\cos^2 \varepsilon - \cos^2 \phi}}{\cos \varepsilon + \sqrt{\cos^2 \varepsilon - \cos^2 \phi}}$$

By comparing the horizontal components (in this case, also the normal components), the different assumptions as to the direction of the total pressure are eliminated.

Column 14 gives the coefficient, H', of the formula: Horizontal component $= H'\ (\frac{1}{2} y h^2)$, in which the general wedge theory is used, taking into account the friction of the wall. ϕ is taken as the angle of internal resistance.

$$H' = \left(\frac{\cos \phi}{n+1}\right)^2 ; \; n = \sqrt{\frac{\sin (\phi + \phi') \sin (\phi - \varepsilon)}{\cos \phi' \cos \varepsilon}}.$$

Column 15 gives the percentage ratio of the area of the theoretical wedge of rupture to the area of the wedge when the surface is horizontal. The plane of rupture was assumed to bisect the angle between the vertical and a line drawn at the angle of internal resistance to the horizontal:

$$A = \tfrac{1}{2} h^2 \frac{\sin \omega \cos \varepsilon}{\sin (90° - \varepsilon - \omega)}, \quad \omega = \text{the angle of the wedge.}$$

In this case, $\phi = 38°$, $\omega = 26$ degrees.

Column 16 gives the percentage ratio of the horizontal component to the horizontal component when the fill is level. The same filling material was used in Test No. 6, and the fill was placed at an angle of 34° above the horizontal, to a depth of 3 ft.

The theoretical coefficients of $(\frac{1}{2} y h^2)$ are:

$H = 0.334$; $C = 0.396$ for the wedge theory, wall friction being considered.
$N = 0.384$ for the wedge theory, wall friction being disregarded.
$K = 0.333$, $R = 0.402$ for the Rankine theory.

The coefficient for the total pressure, corresponding to $N = 0.384$, and $\phi' = 32° 30'$, as in Test No. 5, is 0.455.

For $\phi' = 31°$, as seems to be case in this test, the coefficient is 0.445.

Conclusions.—For a vertical wall, with any inclination of the surface of the fill, the value of the lateral earth pressure is given in:

(*a*) Magnitude, by the formulas derived by the wedge theory for the horizontal component, assuming the pressure to be normal to the wall, that is, disregarding the wall friction. This might be expected, as the previous tests showed that the nature of the wall

TABLE 8.—TEST NO. 5.

(See Fig. 4.)

September 25, 1921.
September 27, 1921.
Weather: Cloudy and rain.
Temperature: 22° cent. (72° Fahr.)
$h = 5$ ft., $a = 0$.

Physical Properties:
Weight per cubic foot = 95 lb.
Angle of Natural Slope = 39° 30′
" " Internal Resistance = 38°
" " Internal Friction = 35° 30′
" " Wall Friction = 32° 30′

Surface slope, ϵ.	Total Pressure, in Pounds.				Inclination of P.	Pressure per Foot, in Pounds.			$\frac{x}{h}$	$\frac{H}{\frac{1}{2}\gamma h^2}$	J.	K.	H'.	Percentage, wedge area.	Percentage, H (Exp.)
	H_1.	H.	V.	P.	ϕ'	H.	V.	P.							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
−33° 40′	255	1064	679	1265	32° 30′	213	136	253	0.33	0.179	0.179	.....	0.163	76	78
−31°	253	1058	678	1260	32° 30′	212	136	252	0.33	0.179	0.183	.....	0.163	77	78
−28° 05′	251	1052	679	1252	33°	210	136	250	0.33	0.177	0.187	.....	0.171	79	77
−25°	251	1052	685	1255	33°	210	137	251	0.33	0.177	0.191	.....	0.176	81	77
−21° 50′	255	1063	689	1270	33°	213	138	254	0.33	0.179	0.196	.....	0.181	84	78
−18° 30′	257	1089	719	1305	33°	218	144	261	0.33	0.184	0.201	.....	0.186	86	80
−14° 55′	258	1101	730	1320	33° 30′	220	146	264	0.33	0.185	0.208	.....	0.192	89	80
−11° 20′	287	1144	778	1385	34°	229	156	277	0.33	0.193	0.213	.....	0.198	93	84
− 7° 40′	299	1229	810	1470	33° 30′	246	162	284	0.33	0.204	0.217	.....	0.205	94	90
− 3° 50′	331	1314	868	1570	33°	263	174	314	0.34	0.221	0.225	.....	0.212	97	96
0	365	1412	914	1680	32° 30′	287	183	336	0.34	0.237	0.237	0.237	0.220	100	100
+ 3° 50′	399	1487	946	1760	33°	297	189	352	0.35	0.250	0.253	0.239	0.228	102	108
+ 7° 40′	416	1547	1011	1850	32° 30′	309	202	370	0.35	0.260	0.264	0.241	0.239	107	113
+11° 20′	517	1679	1080	1945	32° 30′	336	216	389	0.40	0.282	0.273	0.244	0.250	111	123
+14° 55′	532	1712	1100	1990	32° 30′	342	220	398	0.40	0.288	0.285	0.249	0.264	115	125
+18° 30′	569	1833	1170	2160	32° 30′	367	234	432	0.40	0.308	0.296	0.253	0.279	119	134
+21° 50′	608	1939	1240	2305	32° 30′	388	248	461	0.42	0.327	0.314	0.266	0.295	124	142
+25°	680	2079	1330	2480	32° 30′	416	266	496	0.40	0.351	0.332	0.278	0.314	129	152
+28° 05′	796	2319	1500	2610	33°	464	300	550	0.44	0.391	0.352	0.296	0.344	135	169
+31°	804	2344	1530	2650	33°	469	306	556	0.44	0.395	0.391	0.319	0.377	142	208
+34°	834	2504	1660	2900	32° 30′	501	320	594	0.43	0.423	0.425	0.362	0.432	149	216
Averages.										0.257	0.258	.....	0.242	.....	.....

surface had no effect on the horizontal component. The vertical component is equal to the horizontal component multiplied by the coefficient of friction on the wall.

(*b*) Direction, by the angle of friction between the fill and the surface of the wall.

(*c*) Point of application, by the value, 0.33 h to 0.40 h, depending on conditions. These conditions are discussed in later tests.

TABLE 9.—TEST NO. 6.

Date: September 22, 1921.
Weather: Clear.
Temperature: 21° cent. (70° Fahr.)

Physical Properties:
Same as for Test No. 5.
Fill at $\epsilon = 34°$.

h, in inches.	TOTAL PRESSURES, IN POUNDS.				PRESSURES, IN POUNDS PER FOOT.			INCLINATION OF P.	$\frac{x}{h}$	$\frac{H}{\frac{1}{2} \gamma h^2}$	$\frac{P}{\frac{1}{2} \gamma h^2}$
	H_1.	H.	V.	P.	H.	V.	P.	ϕ'			
12	13	112	43	120	24	9	24	....	....	0.53	0.53
18	15.5	168	103	192	33	21	38	32°	0.48	0.33	0.86
24	25	373	211	429	75	42	86	30°	0.28	0.419	0.480
30	40.5	501	300	584	100	60	117	31°	0.33	0.356	0.416
36	102	756	473	891	151	95	178	32°	0.33	0.373	0.440
Averages, 2 ft. to 3 ft.....										0.382	0.444

The point of application of the resultant for fills below the horizontal is at $\frac{1}{3} h$, and is higher for fills above the horizontal, reaching 0.40 h for fills having the greatest slopes. This test proves the presence of a vertical component for all cases. The Rankine theory would give an upward component for all the cases of negative ϵ. Columns 15 and 16 were computed in order to compare the variation of pressure with that of the wedge area. The pressures seem to vary much more than the area of the wedge, especially at the greater slopes. The value of the pressure, therefore, is a function of a number of variables, including the area of the wedge, and not on the area of the wedge itself, as is assumed by some. This is based on the assumption of a plane of rupture which bisects the angle between the vertical and the line drawn at the angle of internal resistance to the horizontal. The general wedge theory furnishes low values, because the friction of the wall is assumed to diminish the pressure.

VI.—Lateral Pressures of Sloping Fills Against a Vertical Glass-Backed Wall.

A few readings were taken of the pressure of a fill, 2 ft. high and inclined at 31° to the horizontal, against a glass-backed vertical wall.

The pressure was 107 lb. per ft. of width, the horizontal component was 92 lb., or a coefficient of 0.500. Theory gives $N = 0.495$, for $\phi_1 = 33° 50'$, $\epsilon =$ 31 degrees. The angle of inclination was about 30°, which is somewhat more than the coefficient of friction on the glass wall, due to the obstructions caused by clamps on the wall. The results agree with the conclusions given in Section V.

VII.—Lateral Pressures of Sloping Fills Against a Sheet-Metal-Backed Vertical Wall.

In order to determine whether the conclusions of Section V were affected by changing the wall, the test for fills above the horizontal was repeated with a sheet-metal wall, the height of the fill being 3 ft.

TABLE 10.—TEST No. 7.

Date: October 9, 1921.
Weather: Clear.
Temperature = 17° cent. (63° Fahr.)
Height of fill = 3 ft.
Wall Vertical.

Physical Properties:
Weight per cubic foot = 94 lb.
Angle of Natural Slope = 38°
" Internal Resistance = 34° 30'
" Internal Friction = 32°
" Wall Friction = 28°

Surface slope, ϵ	Total Pressures, in Pounds.				Pressures, in Pounds per Foot.			Inclination of P.	$\frac{x}{h}$	$\frac{H}{\frac{1}{2}\gamma h^2}$
	H_1.	H.	V.	P.	H.	V.	P.	ϕ'		
0	122	585	340	675	117	68	135	30°	0.333	0.280
+ 4° 45'	127	601	350	695	120	70	139	30°	0 339	0.297
+11° 20'	148	669	397	779	134	79	156	30° 30'	0.348	0.316
+16° 40'	167	764	434	884	153	87	177	29° 30'	0.358	0.337
+21° 50'	206	895	495	1 045	175	99	209	29°	0.375	0.366
+26° 30'	237	996	544	1 138	199	109	228	28° 30'	0.402	0.409
+27° 40'	270	1105	592	1 252	221	118	250	28°	0.415	0.423
Averages.....								28° 45'	.....	0.347

The same conclusions may be drawn from the results given in Table 10 as from those of Test No. 6. The resultant acts at the angle of friction on the wall, and above $\frac{1}{3}h$, especially for the greater slopes. The amount of the horizontal component is given by the wedge theory, which disregards the effect of the friction of the wall. Sections V, VI, and VII show that the horizontal component of the pressures of any kind of fill on a vertical wall is independent of the nature of the wall surface.

Table 11 is presented in order that the coefficient of $(\frac{1}{2}\gamma h^2)$ as found experimentally may be compared with those values found by the various theories, for horizontal components.

TABLE 11.

Experiment:.....	0.280	0.297	0.316	0.337	0.366	0.409	0.423
N (wedge theory).....	0.277	0.288	0.311	0.336	0.363	0.403	0.416
K (Rankine theory)	0.277	0.277	0.284	0.296	0.317	0.349	0.362
H (wall friction theory).....	0.220	0.228	0.254	0.279	0.308	0.347	0.363
ϵ (surface slope).....	0	4°45'	11°20'	16°40'	21°50'	26°30'	27°40'

VIII.—Lateral Pressures of Horizontal Fills Against Wooden Walls with Positive Back Batter.

In this test, four walls were used, with backs sloping at 1 : 12 (4° 45'), 1 : 8 (7° 15'), 1 : 6 (9° 30'), and 1 : 4 (14° 00'). The fixed parts of the recording apparatus required the entire test wall to be placed inside the bin. The

desired shape of wall was constructed and nailed to the inside face of the wooden vertical wall. Several of the tests were repeated because sand, leaking through cracks and getting under the wall, decreased the vertical component. In the repeated tests, it was found that this trouble had little effect on the horizontal readings, as the friction under the wall was quite small.

A preliminary test was made, with a 1 : 4 wall, to determine the direction of the resultant. Special care was taken in this test to have the wall free. The fill was about 4 ft. high, but because of several changes in its shape, made to prevent leakage, no attempt to deduce the magnitude of the pressure from the data of this test is made. The height of fill was always measured vertically.

TABLE 12.—Actual Pressures, in Pounds.

	Top horizontal.	Bottom horizontal.	Vertical.	Total horizontal.	$\frac{V}{H}$	$(\phi' + \alpha)$
a.....	87	708	714	795	0.895	41°55′
b.....	85	714	739	799	0.925	42°45′
c.....	85	712	716	797	0.898	41°55′
d.....	81	716	749	797	0.940	43°10′
e.....	83	714	737	797	0.925	42°45′
f.....	83	714	742	797	0.930	42°00′
g.....	84	715	727	799	0.910	42°20′
Average.....					0.918	42°25′

The space beneath the wall was cleaned after each reading. This procedure necessitated raising the wall about $\frac{1}{100}$ in. each time, and lowering it again; however, the readings are quite consistent.

The resultant is inclined to the horizontal at an angle of 42° 25′ and the wall slopes 14° 00′ from the vertical, so that the resultant makes an angle of 28° 25′ to the normal to the wall. The angle of wall friction, as determined experimentally for this sand (93 lb. per cu. ft.; moisture content, 3.2%; $\phi_1 = 31°$), was slightly less than 29 degrees. This proves that on a wall inclined with positive batter the resultant acts at an angle to normal, which angle is equal to the angle of friction between the wall and the fill.

To determine the magnitude and point of application of the resultant, a test was made on each wall, with horizontal fills, and a test with oblique fills (described subsequently) was made on the 1:4 wall.

The readings and the computation of the components and of the total pressure were the same as in the tests with the vertical walls. In addition, the values of the normal and tangential components (N and T) on the wall are given. The relations between the P (total pressure), H (horizontal component), V (vertical component), N, T, ϕ' (angle of wall friction), and α (angle of inclination of the wall from the vertical), are:

$$P = \sqrt{H^2 + V^2}; \qquad \tan(\phi' + \alpha) = \frac{V}{H}$$

$$H = P\cos(\phi' + \alpha);\quad N = P\cos\phi' = \frac{H\cos\phi'}{\cos(\phi' + \alpha)}$$

$$V = P\sin(\phi' + \alpha);\quad T = P\sin\phi' = \frac{H\sin\phi'}{\cos(\phi' + \alpha)}$$

In computing the height of the resultant, the moment of the wall about the vertical supports had to be considered, as the wall was no longer balanced.

The height of the resultant,

$$x = \frac{6.04\ H_1 + V\left(0.48 + \frac{6}{a}\right) + V_0\left(0.48 + \frac{2}{a}\right)}{H + \frac{V}{a}}$$

For taking moments about the vertical supports,

$$H\,x = H_1(6.04) + 0.48\,V + V(6 - x)\frac{1}{a} + V_0\left(0.48 + \frac{6}{3\,a}\right);$$

in which,

H = total horizontal component;
H_1 = top horizontal component with lever arm of 6.04;
V = total vertical component, taken to act at the same point on the back of the wall as H;
V_0 = additional weight of the wall due to false wall;
a = batter ratio $(1 : a)$.

V acts at a distance $\left(0.48 + \frac{6 - x}{a}\right)$ ft. from the axis.

V_0 acts at a distance $\left(0.48 + \frac{6}{3\,a}\right)$ ft. from the axis, assuming the additional piece to be a solid triangular wedge of base $\frac{6}{a}$ ft.

TABLE 13.—TEST No. 8.

Date: November 17, 1921.
Weather: Clear.
Temperature = 14° cent. (57° Fahr.)
Wall Slope = + 1 : 12 (4°45′)
Moisture Content = 3.5%

Physical Properties:
Weight per cubic foot = 96 lb.
Angle of Natural Slope = 32°45′
" " Internal Resistance = 29°40′
" " Internal Friction = 27°30′
" " Wall Friction = 29°.

Height of fill, in feet.	Pressures, per Foot of Wall, in Pounds.					Height of P.	Coefficients of $(\frac{1}{2}\gamma h^2)$		
	H.	V.	P.	N.	T.	$\frac{x}{h}$	H.	P.	N.
2.5	70	47	84	73	44	0.39	0.234	0.280	0.244
3.	92	62	113	98	54	0.39	0.213	0.260	0.215
3.5	125	84	151	132	74	0.405	0.213	0.257	0.225
4.	154	104	187	163	90	0.396	0.203	0.247	0.215
Averages						0.395	0.216	0.271	0.225

$V = H\tan(\phi' + \alpha)$; $T = N\tan(\phi')$.

TABLE 14.—TEST No. 9.

Date: November 20, 1921.
Weather: Clear.
Temperature = 9.5° cent. (49° Fahr.)
Wall Slope = + 1 : 8 (7° 15′)
Moisture Content = 3.5%

Physical Properties:
Weight per cubic foot = 96 lb.
Angle of Natural Slope = 32°15′
" " Internal Resistance = 29°40′
" " Internal Friction = 27°30′
" " Wall Friction = 29°

Height of fill, in feet.	Pressures, per Foot of Wall, in Pounds.					Height of P	Coefficients of $(\frac{1}{2} y h^2)$.		
	$H.$	$V.$	$P.$	$N.$	$T.$	$\frac{x}{h}$	$H.$	$P.$	$N.$
2.5	69	49	85	74	41	0.345	0.230	0.283	0.247
3.	88	64	108	95	53	0.337	0.204	0.250	0.220
3.5	115	84	142	124	69	0.378	0.195	0.241	0.211
4.	148	108	184	160	89	0.420	0.198	0.239	0.208
Averages						0.370	0.208	0.253	0.222

In order to compare the experimental results with the various theories, the following coefficients were computed for each type of wall. The Rankine theory, as originally given, does not apply to a sloped wall. The general wedge theory, assuming the resultant to act at an angle, ϕ', to the normal to the wall, that is $(\phi' + \alpha)$ to the horizontal, gives,

$$P = \tfrac{1}{2} y h^2 (C) \text{ and } H = \tfrac{1}{2} y h^2 (H)$$

for the total and horizontal components. For a level fill,

$$(C) = \left(\frac{\cos(\phi - \alpha)}{(n+1)\cos\alpha}\right)^2 \frac{1}{\cos(\phi' + \alpha)}; \; n = \sqrt{\frac{\sin(\phi + \phi')\sin(\phi)}{\cos(\alpha)\cos(\phi' + \alpha)}};$$

$$(H) = C\cos(\phi' + \alpha).$$

The wedge theory, which assumes the resultant pressure to act normal to the wall, gives $N = \frac{1}{2} y h^2 (N)$ as the normal component, in which,

$$(N) = \left(\frac{\cos(\phi - \alpha)}{(n+1)\cos\alpha}\right)^2 \frac{1}{\cos\alpha} = \left(\frac{\cos(\phi - \alpha)}{\sin\phi + \cos\alpha}\right)^2 \frac{1}{\cos\alpha}; \; n = \frac{\sin\phi}{\cos\alpha}.$$

TABLE 15.—TEST No. 10.

Date: November 21, 1921.
Weather: Clear.
Temperature = 10° cent. (50° Fahr.)
Wall Slope = + 1:6 (9°30′)
Moisture Content = 3.7%

Physical Properties:
Weight per cubic foot = 97 lb.
Angle of Natural Slope = 36°
" " Internal Resistance = 32°
" " Internal Friction = 29°40′
" " Wall Friction = 29°

Height of fill, in feet.	Pressures, per Foot of Wall, in Pounds.					Height of P.	Coefficients of $(\frac{1}{2} y h^2)$.		
	$H.$	$V.$	$P.$	$N.$	$T.$	$\frac{x}{h}$	$H.$	$P.$	$N.$
2.5	61	48	78	73	34	0.388	0.202	0.254	0.241
3	81	65	104	91	50	0.373	0.186	0.238	0.208
3.5	121	97	155	136	75	0.392	0.204	0.261	0.229
4	157	125	194	176	97	0.405	0.208	0.250	0.227
Averages						0.390	0.199	0.251	0.226

TABLE 16.—Test No. 11.

Date: November 23, 1921.
Weather: Clear.
Temperature = 14° cent. (57° Fahr.)
Wall Slope = + 1:4 (14°00′)
Moisture Content = 3.7%

Physical Properties:
Weight per cubic foot = 97 lb.
Angle of Natural Slope = 35°
" " Internal Resistance = 31°
" " Internal Friction = 24°15′
" " Wall Friction = 29°

Height of fill, in feet.	Pressures, per Foot of Wall, in Pounds.					Height of P.	Coefficients of $(\frac{1}{2} \gamma h^2)$.		
	H.	V.	P.	N.	T.	$\frac{x}{h}$.	H.	P.	N.
2.5	93	86	127	111	61	0.390	0.308	0.420	0.367
3	125	117	172	150	83	0.388	0.287	0.394	0.344
3.5	152	142	209	182	101	0.409	0.256	0.352	0.306
Averages						0.396	0.284	0.389	0.339

The ratio between the experimental and theoretical values of $H = 0.790$; of $C = 0.795$; and of $N = 0.65$; therefore, the tests seem to show that the general wedge theory, taking into account the wall friction, applies to battered walls and a horizontal fill within 20 per cent. The resultant is inclined to the normal at an angle equal to that of wall friction. The resultant acts at about $\frac{2}{3} h$. The error in these tests seems to be quite large, unless all theories are disregarded as giving too high results. The writer does not believe that the error of these tests is greater than 10% in any one case.

TABLE 17.—Comparison of Experiment with Theory—Inclined Walls.
(See Fig. 5, Horizontal Fill, Heel of Wall in Fill.)

Slope of wall.	Experimental.						Theoretical.				Experiment.	
	α	ϕ'	γ.	H.	C.	N.	C.	H.	N.	J.	$\frac{x}{h}$	ϕ
+1:12	4° 45′	29°	96	0.216	0.271	0.225	0.338	0.281	0.371	0.370	0.395	29° 40′
+1: 8	7° 15′	29°	96	0.208	0.253	0.222	0.360	0.290	0.383	0.380	0.370	29° 40′
+1: 6	9° 30′	29°	97	0.199	0.251	0.226	0.255	0.278	0.374	0.368	0.390	32° 00′
+1: 4	14° 00′	29°	97	0.284	0.389	0.339	0.413	0.302	0.428	0.415	0.396	31° 00′
Averages				0.227	0.291	0.253	0.367	0.288	0.389	0.383	0.388	.....

IX.—Lateral Pressure of Horizontal Fills Against a Wooden Wall with Negative Back Batter.

In this test, five walls were used with backs sloping at 1 : 12 (4° 45′), 1 : 6 (9° 30′), 1 : 4 (14° 00′), 1 : 3 (18° 25′), and 1 : 2 (27° 15′). The wall was constructed in the same manner as that described in Section VIII. As the lower end of the wall was an edge, the upper end overhanging, there was no trouble in attaching the extra wall to the original vertical wall and no possibility of sand leaking under the wall. These tests are, therefore, more accurate.

The readings and the computation of the pressures are the same as for the vertical walls. In computing the height of the resultant, the moment of the

wall about the vertical supports had to be taken into account, because the wall was not absolutely balanced. The height of the resultant, x, is given by the formula:

$$x = \frac{6.04\ H_1 + 0.48\ V + V_0\left(0.48 + \frac{4}{3\ a}\right)}{H - \frac{V}{a}}$$

The derivation of this formula is the same as that given in Section VIII.

TABLE 18.—TEST NO. 12.

Date : October 30, 1921.
Weather : Clear.
Temperature : 18° cent. (65° Fahr.)
Wall Slope = —1:12, (4° 45′); $(\phi' + \alpha) = 25°$.
Moisture Content = 3.65%

Physical Properties:
Weight per cubic foot = 97 lb.
Angle of Natural Slope = 34° 50′
" Internal Resistance = 30° 30′
" Internal Friction = 27° 55′
" Wall Friction = 29° 45′

Height of fill, h, in feet.	H_1.	Total Pressures, in Pounds.			$(\phi' + \alpha)$	Pressures per Foot.			$\frac{x}{h}$	$\frac{H}{\frac{1}{2} y h^2}$	$\frac{P}{\frac{1}{2} y h^2}$
		H.	V.	P.		H.	V.	P.			
2	45	204	84	220	22°	41	17	44	0.370	0.216	0.232
2.5	67	298	127	308	23°	60	25	62	0.344	0.201	0.208
3	105	458	207	503	24°	92	41	101	0.343	0.214	0.235
3.5	152	639	307	710	26°	128	61	142	0.340	0.218	0.246
4	219	892	441	995	26° 20′	178	88	199	0.377	0.234	0.262
4.5	238	1067	532	1190	26° 30′	213	106	238	0.375	0.222	0.248
Averages					24° 38′				0.358	0.221	0.239

TABLE 19.—TEST NO. 13.

Date : November 6, 1921.
Weather : Clear.
Temperature = 14° cent. (57° Fahr.)
Wall Slope = —1 : 6, (9° 30′); $(\phi' + \alpha) = 19° 30'$.
Moisture Content = 3.25%

Physical Properties:
Weight per cubic foot = 94 lb.
Angle of Natural Slope = 38° 20′
" Internal Resistance = 31° 50′
" Internal Friction = 28° 30′
" Wall Friction = 29° +

Height of fill, h, in feet.	H_1.	Total Pressures, in Pounds.			$(\phi' + \alpha)$	Pressures per Foot.			$\frac{x}{h}$	$\frac{H}{\frac{1}{2} y h^2}$	$\frac{P}{\frac{1}{2} y h^2}$
		H.	V.	P.		H.	V.	P.			
2.5	26	306	91	320	16° 30′	61	18	64	0.40	0.208	0.218
3	43	424	139	445	18° 10′	85	28	89	0.35	0.201	0.210
3.5	74	565	199	599	19° 20′	113	40	120	0.34	0.196	0.208
4	126	645	256	695	21° 40′	129	51	139	0.40	0.172	0.185
Averages					19° 00′				0.37	0.195	0.205

It was noticed that the sand seemed to "creep" away from the wall. There seemed to be a space, between the sand and the wall, at least 1 ft. deep and about ½ in. wide. As the depth of fill increased, the lower part probably had a better grip on the wall, hence the increase in the value of $(\phi' + \alpha)$.

TABLE 20.—TEST NO. 14.

Date: November 10, 1921.
Weather: Clear.
Temperature = 7.5° cent. (47° Fahr.)
Wall Slope = −1 : 4, (14°); $(\phi' + \alpha) = 13°$.
Moisture Content = 3.5%.

Physical Properties:
Weight per cubic foot = 97 lb.
Angle of Internal Resistance = 31°.
" Natural Slope = 34°.
" Internal Friction = 29°.
" Wall Friction = 27°.

Height of fill, h, in feet.	H_1.	Total Pressures, in Pounds.			$(\phi' + \alpha)$.	Pressures per Foot.			$\frac{x}{h}$	$\frac{H}{\frac{1}{2} y h^2}$	$\frac{P}{\frac{1}{2} y h^2}$
		H.	V.	P.		H.	V.	P			
2.5	20	260	58	266	12° 25′	52	12	53	0.40	0.173	0.177
3	39	372	84	380	12° 35′	74	17	76	0.32	0.167	0.171
3.5	88	545	123	560	12° 45′	109	25	112	0.36	0.185	0.190
4	134	695	154	710	12° 30′	139	31	142	0.36	0.183	0.187
Averages.....					12° 34′	.....	.....	.....	0.36	0.177	0.181

TABLE 21.—TEST NO. 15.

Date: November 11, 1921.
Weather: Cloudy and Rain.
Temperature = 8° cent. (46° Fahr.)
Wall Slope = −1 : 3, (18° 25′); $(\phi' + \alpha) = 11° 35'$.
Moisture Content = 3.55%

Physical Properties:
Weight per cubic foot = 97 lb.
Angle of Internal Resistance = 30° 20′
" Natural Slope = 36° 20′
" Internal Friction = 27° 30′
" Wall Friction = 30°

Height of fill, h, in feet.	H_1.	Total Pressures, in Pounds.			$(\phi' + \alpha)$	Pressures per Foot.			$\frac{x}{h}$	$\frac{H}{\frac{1}{2} y h^2}$	$\frac{P}{\frac{1}{2} y h^2}$
		H.	V.	P.		H.	V.	P.			
2.5	18	269	67	276	14°	54	13	55	0.40	0.181	0.185
3	38	365	80	373	12° 20′	73	16	75	0.39	0.171	0.175
3.5	79	488	101	499	11° 40′	98	20	100	0.39	0.167	0.170
4	122	619	128	630	11° 10′	124	26	126	0.40	0.163	0.166
Averages.....					12° 53′	.....	.....	.....	0.395	0.171	0.174

TABLE 22.—TEST NO. 16.

Date: November 13, 1921.
Weather: Cloudy.
Temperature = 6.5° cent. (44° Fahr.)
Wall Slope = −24.75 : 48, (27° 15′); $(\phi' + \alpha) = 2° 45'$.
Moisture Content = 3.55%.

Physical Properties:
Weight per cubic foot = 97 lb.
Angle of Internal Resistance = 30° 20′
" Natural Slope = 36° 30′
" Internal Friction = 27° 30′
" Wall Friction = 30°

Height of fill, h, in feet.	H_1.	Total Pressures, in Pounds.			$(\phi' + \alpha)$	Pressures per Foot.			$\frac{x}{h}$	$\frac{H}{\frac{1}{2} y h^2}$	$\frac{P}{\frac{1}{2} y h^2}$
		H.	V.	P.		H.	V.	P.			
2.5	11	184	14	185	4° 30′	37	3	37	0.40	0.124	0.124
3	16	235	21	236	5°	47	4	47	0.35	0.110	0.110
3.5	30	300	29	301	5° 30′	60	6	60	0.40	0.102	0.102
4	55	378	41	381	8° 20′	76	8	76	0.40	0.100	0.100
Averages.....					5° 50′	.....	.....	.....	0.39	0.109	0.109

In order to compare the experimental results with the various theories, the coefficients were computed as in Section VIII. The formulas are the same, but the value of α is now negative. Instead of the normal pressure, N, the horizontal component, J, of the normal pressure is given: $J = N \cos \phi'$.

TABLE 23.—Comparison of Experiment with Theory. Inclined Walls.
(See Fig. 5. Horizontal Fill. Top of Wall Overhanging.)

Slope of wall.	ϕ	ϕ'	y	Direction of P.		Experiment.		Theory.			Experiment, $\frac{x}{h}$
				Experiment.	Theory.	H.	P.	H.	P.	J.	
1 : 12	30° 30′	29° 45′	97	24° 38′	25°	0.221	0.239	0.229	0.256	0.295	0.358
1 : 6	31° 50′	29°	94	19° 00′	19° 30′	0.195	0.205	0.197	0.210	0.245	0.37
1 : 4	31° 00′	29°	97	12° 34′	13°	0.177	0.181	0.189	0.194	0.226	0.36
1 : 3	30° 20′	30°	97	12° 53′	11° 35′	0.171	0.174	0.172	0.176	0.208	0.395
1 : 2	30° 20′	30°	97	5° 50′	2° 45′	0.109	0.109	0.113	0.113	0.144	0.39
Averages.....				14° 59′	14° 22′	0.175	0.182	0.180	0.190	0.224	0.375

The total pressure is given very closely by the general wedge formula, taking into consideration the wall friction. Its direction is always at an angle, ϕ', to the normal, and its height at about $\frac{2}{3}\ h$. The height of application seems to rise with an increase in the inclination of the wall.

X.—Lateral Pressure of Sloping Fills Against a Wall with a Positive Back Batter of 1 : 4.

As in the tests with a vertical wall, various surfaces, with a constant height of wall of 3 ft., were tested. For the method of computation, see Section VIII. In calculating the theoretical values, the following formulas were used:

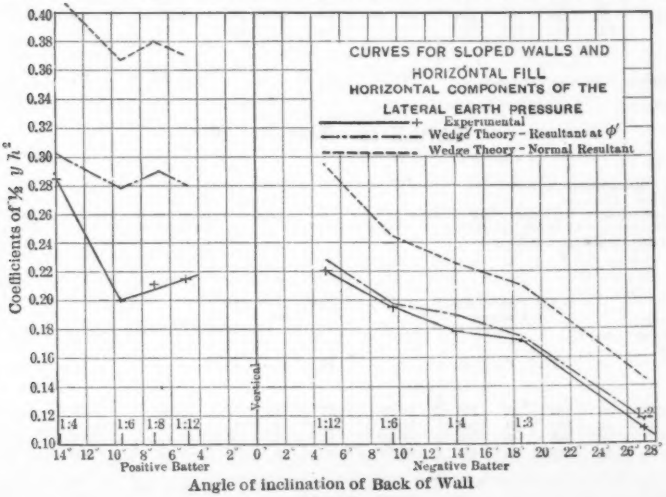


Fig. 5.

The general wedge theory, taking into consideration the wall friction, is:

$$E = \tfrac{1}{2}\, y\, h^2 C,\quad C = \left(\frac{\cos(\phi - \alpha)}{(n+1)\cos\alpha}\right)^2 \frac{1}{\cos(\phi' + \alpha)},$$

$$n = \sqrt{\frac{\sin(\phi + \phi')\sin(\phi - \varepsilon)}{\cos(\phi' + \alpha)\cos(\alpha - \varepsilon)}}$$

The horizontal component coefficient, $H = C\cos(\phi' + \alpha)$.

TABLE 24.—Test No. 17.
(See Fig. 6.)

Date: November 24, 1921.
Weather: Clear.
Temperature = 14° cent. (57° Fahr.)
Wall Slope = 1 : 4, (14°).
Moisture Content = 3.7%.

Physical Properties:
Weight per cubic foot = 93 lb.
Angle of Natural Slope = 35°
" Internal Resistance = 81°
" Internal Friction = 29° 15'
" Wall Friction = 29°

$(\phi' + \alpha) = 43°$. $\tfrac{1}{2}\, y\, h^2 = 419$

Surface slope. ϵ	Pressures, in Pounds per Foot.					Height of P.	Coefficients of $\tfrac{1}{2}\, y\, h^2$.					
							Experimental.			Theoretical.		
	H.	V.	P.	N.	T.	$\frac{x}{h}$	H.	P.	N.	H.	N.	J.
	63	58	86	75	42	0.33	0.151	0.206	0.179	0.168	0.266	0.258
	77	72	106	93	50	0.33	0.184	0.253	0.222	0.195	0.299	0.290
	90	84	124	108	60	0.34	0.215	0.296	0.258	0.235	0.351	0.340
	100	94	139	120	66	0.34	0.239	0.332	0.287	0.255	0.373	0.362
	110	102	150	131	73	0.35	0.263	0.358	0.313	0.276	0.400	0.388
	124	115	170	148	82	0.376	0.296	0.405	0.354	0.302	0.428	0.415
	132	123	180	157	87	0.396	0.315	0.430	0.375	0.336	0.464	0.450
	140	131	192	168	93	0.415	0.335	0.459	0.401	0.380	0.514	0.494
	155	145	213	186	103	0.416	0.370	0.509	0.445	0.425	0.557	0.540
	160	149	219	191	106	0.420	0.382	0.523	0.456	0.451	0.583	0.565
	166	155	228	199	110	0.439	0.397	0.545	0.474	0.571	0.685	0.665
	178	166	244	213	118	0.446	0.425	0.583	0.510	0.765	0.863	0.837
.....							0.298	0.408	0.357	0.363	0.482	.....

The normal pressure wedge theory, disregarding the wall friction, is:

$$E = \tfrac{1}{2}\, y\, h^2 N,\quad N = \left(\frac{\cos(\phi - \alpha)}{(n+1)\cos\alpha}\right)^2 \frac{1}{\cos\alpha};\quad n = \sqrt{\frac{\sin\phi\sin(\phi - \varepsilon)}{\cos\alpha\cos(\alpha - \varepsilon)}}$$

The direction of P is $\cos^{-1}\dfrac{H}{P} = \dfrac{298}{408} = 0.732$, or 43°.

The point of application varies from $\frac{1}{3}\ h$ to $\frac{1}{2}\ h$, depending on the slope. The experimental value of the resultant is 82% of the theoretical, as given by the general wedge theory, and 74% of the normal pressure wedge theory.

XI.—Lateral Pressure of Sloping Fills Against a Wall with a Negative Back Batter of 1 : 6.

A test similar to the previous one was made with a wall having an overhanging top, that is, a negative batter. The computations and theoretical formulas are the same as in Sections IX and X.

The direction of the resultant to the horizontal is $(\phi' + \alpha)$, or at an angle, ϕ', to the normal. The point of application varies, but is always above the third point and rises with increased slopes of fill. The average magnitude of the

resultant is given very accurately by the general wedge theory, the theoretical values being about 5% lower than the experimental values, except for the very steep slopes above the horizontal, in which case the experimental are much lower than the theoretical values.

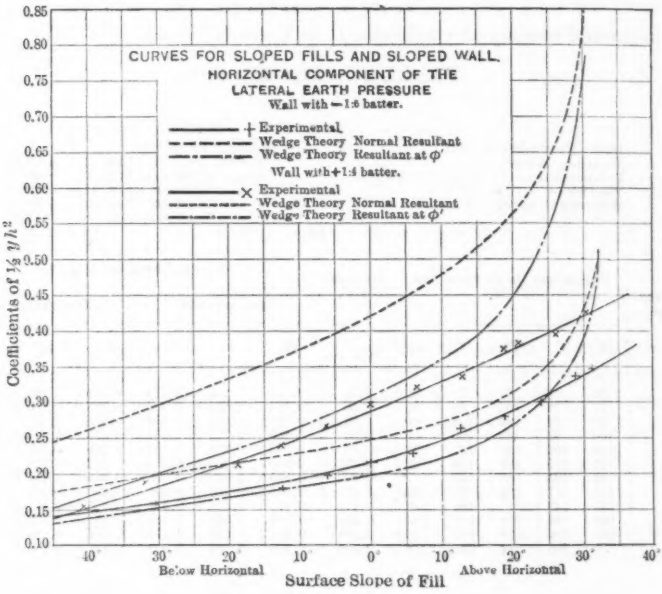


FIG. 6.

XII.—Lateral Pressure of Irregular Fills Against a Vertical Wooden Wall.

This test was made to determine the presence or absence of a wedge of rupture. The bin was filled to a level of 6 ft. and several readings were taken. Starting at the back, 9 ft. from the wall, the fill was raised 1 ft. Readings were taken as the increased height of fill approached the wall. In this manner, the rate of increase in lateral pressure was determined. If there is a fixed prism of rupture, the increase in fill beyond the plane of rupture will have no effect on the lateral pressure, but an increase in fill between the wall and the plane of rupture will cause an increase in pressure. The increase in fill was governed by placing a board across the bin at the required distance from the back, 1 ft., 2 ft., etc., successively. Sand was shoveled behind this board to a height of 1 ft., and readings were taken. This procedure practically corresponded to a uniform load of 100 lb. per sq. ft. moving toward the wall. After these readings were taken, the board was removed, which allowed the sand to take a position of natural repose. The distance from the wall to the toe of the slope was about 6 in. less than the distance from the wall to the previous vertical face of the increased fill. Readings were taken after this change.

The report of the test in Table 26 gives the horizontal and vertical components and the resultant pressure per foot of width of wall for each increment of fill in both shapes, together with the percentage increases in each value. Assuming the plane of repose to be 36° to the horizontal, and the plane of rupture to bisect the angle between the vertical wall and the plane of repose, the areas of the wedges between the wall and the two inclined planes were computed. These values include the additional fill. The percentage increases in these values are also given.

TABLE 25.—TEST NO. 18.
(See Fig. 6.)

Date: November 8, 1921.
Weather = Clear.
Temperature = 12° cent. (54° Fahr.)
Wall slope = −1 : 6, (9° 30′).
Moisture Content = 3.3%

Physical Properties:
Weight per cubic foot = 94 lb.
Angle of Natural Slope = 38° 20′
" " Internal Resistance = 32°
" " Internal Friction = 28° 20′
" " Wall Friction = 30°

h = 3 ft. $(\phi' + \alpha) = 20° 30'$. $\frac{1}{2}\gamma h^2 = 423$.

Surface slope, ϵ		Pressures per Foot, in Pounds.			Inclination of P.	$\frac{x}{h}$	$\frac{H}{\frac{1}{2}\gamma h^2}$	$\frac{P}{\frac{1}{2}\gamma h^2}$	Theoretical Coefficients.		
		H.	V.	P.	$(\phi' + \alpha)$				H.	C.	J.
−38° 20′	19 : 24	63	23	67	20° 00′	0.398	0.149	0.158	0.139	0.148	0.187
−30° 15′	1 : 2	67	24	72	19° 40′	0.382	0.158	0.170	0.153	0.164	0.198
−12° 30′	2 : 9	76	26	80	19° 10′	0.353	0.180	0.189	0.176	0.187	0.223
− 6° 20′	1 : 9	82	31	87	20° 50′	0.358	0.194	0.205	0.186	0.199	0.233
Level	.....	91	33	97	19° 55′	0.358	0.215	0.229	0.198	0.211	0.245
6° 20′	1 : 9	98	36	104	20° 10′	0.385	0.231	0.246	0.211	0.225	0.260
12° 30′	2 : 9	109	40	117	20° 20′	0.365	0.258	0.277	0.232	0.248	0.280
18° 25′	1 : 3	117	44	125	20° 40′	0.374	0.276	0.296	0.258	0.276	0.305
24° 00′	4 : 9	128	48	136	20° 40′	0.385	0.300	0.321	0.297	0.317	0.343
29° 00′	5 : 9	141	54	151	21° 00′	0.368	0.333	0.357	0.369	0.394	0.407
31° 30′	11 : 18	147	57	158	21° 10′	0.370	0.347	0.374	0.475	0.507	0.495
Averages.....					20° 20′	.....	0.240	0.257	0.245	0.261	0.289

Conclusions.—The test shows the presence of a "wedge" or "prism" of rupture, closely approximating the theoretical wedge. Loading beyond the theoretical plane of rupture had practically no effect on the pressure. As soon as the load crossed the plane of rupture, the horizontal and vertical components increased, the vertical somewhat faster than the horizontal. The variation in the total pressure is very close to that in the theoretical wedge of rupture. The height of the resultant remained practically unchanged. It is evident that all the material above the plane of repose does not act as the wedge of rupture. Whether or not the surface of rupture is a plane, is difficult to determine. There was no indication of a break in the top surface of the fill in any of the tests. Such a break could hardly be expected from the small movements of the wall.

XIII.—Effect of Settling and Changes of Temperature on the Lateral Pressure of Horizontal Fills Against a Vertical Wall.

Several tests were conducted to determine the effect of changes in temperature, humidity, etc., and to investigate the effect of settling. Readings were taken about 1 hour apart. One test consisted in taking readings for a period

of 8 days, during which there was a temperature variation of 13° cent. (23° Fahr.), and several rains.

TABLE 26.—TEST No. 19.

Date: May 19, 1921
Weather: Clear.
Temperature: 28° cent. (82° Fahr.)

Physical Properties: Same as in Test No. 1.
Original Fill: 6 ft., level.

Additional fill 1 ft. high.	Distance from wall, in feet.	Pressures, per Foot, in Pounds.			Height of P.		Percentage Increases.						
		H.	V.	P.	$\frac{V}{H}$.	$\frac{x}{h}$.	H.	V.	P.	r.*	u.†	$\frac{x}{h}$.	$\frac{V}{H}$.
None	....	467	296	552	0.633	0.368	0.0	0	0	0	0.0	0	0.0
Vert	8	468	296	554	0.633	0.367	0.2	0	0	3.5	0	0	0
Sloped	7.5	468	297	555	0.633	0.367	0.2	0	1	3.5	0	0	0
Vert	7	468	297	555	0.633	0.367	0.2	0	1	7.0	0	0	0
Sloped	6.5	468	297	555	0.633	0.367	0.2	0	1	7.0	0	0	0
Vert	6	470	297	556	0.633	0.368	1.0	0	1	11.0	0	0	0
Sloped	5.5	470	298	556	0.633	0.368	1.0	1	1	11.0	0	0	0
Vert	5	471	299	558	0.634	0.369	1.0	1	1	15	0	0	0
Sloped	4.5	471	299	558	0.634	0.369	1.0	1	1	15	0	0	0
Vert	4	473	301	561	0.635	0.369	1	2	2	19	0	0	0
Sloped	3.5	473	301	561	0.636	0.370	1	2	2	19	0	0	0
Vert	3	481	316	576	0.655	0.369	3	7	4	23	3.5	0	3.5
Sloped	2.5	484	320	580	0.662	0.368	4	8	5	23	3.5	0	4.5
Vert	2	499	341	605	0.684	0.361	7	15	9	27	14.5	−2	8
Sloped	1.5	500	341	605	0.682	0.360	7	15	9	27	14.5	−2	8
Sloped	0	569	374	680	0.656	0.380	22	26	23	36	28	+3	4
Sums							50	78	61	250	64	.....	28

* Area between wall and plane of repose for 6-ft. level: 24.78 sq. ft.
† Area between wall and plane of rupture for 6-ft. level: 9.18 sq. feet.

A "blank test", in which readings on the scales were caused by a mechanically applied pressure against the walls, showed very little variation, thereby proving that the variations in the following tests are due to changes in the fill and not to changes in the apparatus. In these tests, only those readings are given in which changes occurred. During the 8-day test mentioned previously, 80 sets of readings, distributed in such a manner that the effect of extremes of temperature and humidity could be investigated, were taken.

The maximum (x) and minimum (n) values are given in Table 27.

TABLE 27.

Age of fill, in hours.	Time of day.	H.	V.	P.	$\frac{V}{H}$	$\frac{x}{h}$
0	5 P.M.	456	229 (x)	545	0.658	0.355 (n)
14	12 M.	419 (n)	286	508 (n)	0.680 (x)	0.363
88–90	9 A.M.	465	278 (n)	542	0.597	0.370
90	11 A.M.	468	278	544	0.594 (n)	0.371
114	11 A.M.	473 (x)	286	553 (x)	0.603	0.374 (x)
Percentage variations:						
Of maximum		3.7	0	1.5	3.3	5.4
Of minimum		8.1	6.3	6.8	9.7	0
Total variations		11.8	6.3	8.3	13.0	5.4

TABLE 28.—Test No. 20, Time Test.

Date: May 12-19, 1921.
Wooden wall: vertical.
Horizontal fill: 6 ft. high.

Physical Properties; Same as in Test No. 1.

Day.	Hour.	Age of fill, in hours.	Temperature, in degrees, centigrade.	Pressures per Foot, in Pounds.			Tan $\phi' = \frac{V}{H}$	$\frac{x}{h}$	Percentage Changes.				Weather and humidity.
				H.	*V.*	*P.*			*H.*	*V.*	*P.*	$\frac{x}{h}$	
1st.....	5 P.M.	0	12	456	299	545	0.658	0 355	.....	.....	.....	.....	Rain.
	12 M.	19	14	419	286	508	0.680	0.363	−8.1	−3.9	−6.8	+2.3	Storm.
2d.....	2 P.M.	21	12	422	286	509	0.678	0.364	−7.4	−3.9	−6.6	+2.5	Clear.
	6 P.M.	25	12	422	286	509	0.679	0.361	−7.4	−3.9	−6.6	+1.7	"
	9 A.M	40	19	426	280	510	0.660	0.363	−6.6	−5.7	−6.4	+2.3	"
3d.....	12 M.	43	20	432	284	517	0.658	0.365	−5.3	−4.5	−5.1	+2.8	"
	4 P.M.	47	20	432	286	518	0.662	0.364	−5.3	−3.9	−5.0	+2.5	"
	11 A.M.	66	16	450	279	540	0.620	0.364	−1.3	−6.0	−2.8	+2.5	"
4th.....	12 M.	67	17	451	279	530	0.620	0.365	−1.1	−6.0	−2.8	+2.8	"
	12:30P.M	67.5	17	453	279	532	0.616	0.365	−0.7	−6.0	−2.4	+2.8	"
	8 A.M.	87	13	460	279	538	0.607	0.366	+0.9	−6.0	−1.3	+3.1	"
	9 A.M.	88	15	465	278	542	0.597	0.370	+2.0	−6.3	−0.6	+4.2	"
	10 A.M.	89	15	466	278	543	0.599	0.371	+2.2	−6.3	−0.4	+4.5	"
	11 A.M.	90	17	468	278	544	0.594	0.371	+2.6	−6.3	−0.2	+4.5	"
	12 M.	91	17	469	280	545	0.595	0.372	+2.8	−5.7	0.0	+4.8	"
5th.....	1 P.M.	92	17.5	469	280	547	0.596	0.372	+2.8	−5.7	+0.4	+4.8	"
	2 P.M.	93	17.5	469	282	547	0.599	0.371	+2.8	−5.1	+0.4	+4.5	"
	3 P.M.	94	17.5	469	282	547	0.601	0.370	+2.8	−5.1	+0.4	+4.2	"
	5 P.M.	96	17	467	283	546	0.606	0.370	+2.4	−4.8	+0.2	+4.2	"
	6 P.M.	97	16.5	466	283	545	0.608	0.370	+2.2	−4.8	0.0	+4.2	"
	10 P.M.	101	14	469	283	548	0.604	0.369	+2.8	−4.8	+0.6	+4.0	"
	8 A.M.	111	14	461	281	540	0.608	0.369	+1.1	−5.4	−0.9	+4.0	"
	9 A.M.	112	16	464	280	541	0.605	0.371	+1.5	−5.7	−0.7	+4.5	"
	10 A.M.	113	17	466	282	545	0.605	0.373	+2.2	−5.1	0.0	+5.1	"
6th.....	11 A.M.	114	19	473	286	553	0.603	0.374	+3.7	−3.9	+1.5	+5.4	"
	12 M.	115	19	471	288	552	0.612	0.374	+3.3	−3.3	+1.3	+5.4	"
	2 P.M.	117	19	469	290	551	0.617	0.373	+2.8	−2.7	+1.1	+5.1	"
	5 P.M.	120	18	465	292	549	0.628	0.372	+2.0	−2.1	+0.7	+4.8	"
	8 A.M.	135	17	450	287	534	0.638	0.368	−1.3	−3.6	−2.0	+3.1	"
7th.....	12 M.	139	20.5	452	289	536	0.640	0.372	−0.9	−3.0	−1.7	+4.8	"
	8 P.M.	147	20	452	294	539	0.650	0.366	−0.9	−1.5	−1.1	+3.1	"
	8 A.M.	159	19	443	292	530	0.659	0.363	−2.8	−2.1	−2.8	+2.3	"
8th.....	11 A.M.	162	23	446	291	533	0.652	0.366	−2.2	−2.4	−2.2	+3.1	"
	12 M.	163	23.5	451	291	534	0.650	0 366	−1.8	−2.4	−2.0	+3.1	"
	2 P.M.	165	26	451	294	539	0.651	0.364	−1.1	−1.5	−1.1	+2.5	Warm.

Conclusions.—The conclusions from this test may be stated as follows:

1.—The pressure just after filling is practically the maximum.

2.—A rise in temperature is accompanied by an increase in both components, and an increase in the height of the resultant.

3.—A drop in temperature is accompanied by a decrease in both components and a decrease in the height of the resultant.

4.—The ratio of vertical to horizontal components seems to be unaffected by the age of the fill or by changes in temperature. It is, however, affected by the humidity because of changes in the water content.

5.—The height of the resultant varies as just noted, and also slowly rises as the age of the fill increases. For a fill with a horizontal top surface, the height of the resultant seems to be a maximum at $\frac{3}{4}\ h$.

6.—Settling of fill is accompanied by a slight decrease in pressure, with intermittent sudden increases, due to small ruptures in the fill. These increases, however, soon vanish.

7.—The minimum pressures occur soon after the filling. (Tests described subsequently show this more clearly than those given in this Section).

8.—Curves plotted for this test give an average variation of the pressure of 0.15% per degree centigrade, taking the average for each day.

The curves of "pressure with time" are similar to the curve of "damped vibrations". The total variation in the horizontal component of 11.8% is worthy of note. A short report of the preliminary tests on the effect of surcharges has been published.* It may be of interest to note that this article, as well as the preliminary description of the apparatus, have appeared in German technical periodicals.†

TABLE 29.—TEST NO. 21, TIME TEST.

Date: May 19-23, 1921
Weather : Clear.
Time test on a tamped fill; 7 ft. high; 6-ft. vertical wall. The fill was 168 hours old when the test began.

Physical Properties: Same as Test No. 1.

Day.	Hour.	Age of fill, in hours.	Temperature, in degrees, centigrade.	Pressures per Foot, in Pounds.			Tan $\phi' = \frac{V}{H}$.	$\frac{x}{h}$	Percentage Changes in:			
				H.	*V.*	P.			*H.*	*V.*	*P.*	$\frac{x}{h}$
1st...	4 P.M.	0	26	569	374	680	0.659	0.380	.....	.....	.....	.....
	5 P.M.	1	25	570	378	684	0.664	0.381	+0.2	+1.1	+0.6	+0.3
2d....	8 A.M.	16	21	560	373	682	0.665	0.383	−1.6	−0.3	+0.3	+0.8
	12 M.	20	24	566	373	678	0.659	0.385	−0.5	−0.3	−0.3	+1.3
	1 P.M.	21	25	565	373	677	0.660	0.385	−0.7	−0.3	−0.4	+1.3
	2 P.M.	22	26	561	375	682	0.658	0.379	−1.4	+0.3	+0.3	−0.3
	5 P.M.	25	25	583	376	690	0.645	0.379	+2.5	+0.5	+1.5	−0.3
3d....	10 A.M.	42	25	568	371	678	0.654	0.378	−0.2	−0.8	−0.3	−0.5
	1 P.M.	45	25	569	370	678	0.651	0.377	0	−1.1	−0.3	−0.8
5th...	1 P.M.	93	27	563	366	671	0.651	0.377	−1.1	−2.1	−1.3	−0.8

The variation in tamped fills seems to be very much less than in ordinary fills. The temperature range in this test was only 5° cent. (9° Fahr.), so that large variations cannot be expected.

TABLE 30.—TIME TESTS.

Test No. 22*a*.
Temperature : 20° cent. (68° Fahr.)
Fill is 5 ft. high, top surface at + 31°. Vertical wooden wall.

Age of fill, in hours.	H_1	Total Pressures, in Pounds.		Weather.
		H.	*V.*	
0	834	2 504	1 458	Clear.
¼	842	2 539	1 509	"
½	844	2 545	1 522	"
¾	846	2 552	1 532	"
1	845	2 552	1 535	"
1¼	845	2 551	1 536	"
1¾	845	2 554	1 541	"
2	844	2 552	1 541	"

Test No. 22*b*.
Temperature : 20° cent. (68° Fahr.)
Fill is 7 ft. 9 in. high against a vertical wooden wall, 6 ft. high.

Age of fill, in hours.	H_1	Total Pressures, in Pounds.		Weather.
		H.	*V.*	
0	1 100	2 962	2 033	Clear.
1	1 114	3 021	2 036	"
1¼	1 108	2 991	2 013	"
43¼	1 088	2 960	2 004	Rain
67	1 079	2 890	1 987	"
73	1 098	2 922	1 993	Cloudy.
.....	.....	.....	.....	.....
.....	.....	.....	.....	.....

* *Engineering News-Record*, January 19, 1922.
† *Der Bauingenieur* (Leipzig), March 15, 1922; *Der Eisenbau* (Berlin), July 25, 1922.

Conclusions.—The conclusions from this test are as follows: The maximum pressure occurs soon after filling. The fill undergoes a settling as soon as it is placed, causing an increase in pressure in about 1 hour. The pressure then slowly decreases, reaching a low point within 24 hours. It then oscillates each day, as shown in the previous tests.

XIV.—Effect of Static Loads on the Lateral Pressure of Horizontal Fills Against a Vertical Wall.

Forty-five square boards, $11\frac{3}{4}$ by $11\frac{3}{4}$ by 1 in., were used to cover the surface of a 6-ft. horizontal fill of sand. A keg of nails weighing 100 lb. was placed on each square, starting with the one farthest from the wall. Readings were taken while the load rested on the fill and after it had been removed. The effect was that of a localized surcharge of 100 lb. per sq. ft. The averages of the five positions of the load at each distance back of the wall gave the values shown in Table 31. The readings for the five positions of the load at each distance back of the wall were about the same. The surprising feature of this test was that, after the load had been removed, the pressure did not diminish immediately. The fill returned to its original condition several days later.

The theoretical plane of rupture, $\phi = 36°$, cuts the top surface at 3.06 ft. The plane of slope cuts the top surface at 8.25 ft.

Conclusions.—There is an appreciable increase in pressure due to loads not on the wedge of rupture. The increase is much more rapid, however, as the load moves on the theoretical wedge. The effect of a static load is to change the physical characteristics of the fill, as the changed pressure continued after the load had been removed. Assuming that the unit weight has not been changed throughout the fill, and is 100 lb. per cu. ft., the final horizontal component requires a ϕ of 35° 10′. Assuming that the concentrated load compresses the entire fill to $\frac{7}{6}$ of its former weight, or 116.7 lb. per cu. ft., ϕ is then 38° 40′. Assuming that the final density is the average of these two extreme values, or 108.3 lb. per cu. ft., ϕ is then 37° 00′. The formula used is $H = \frac{1}{2} y h^2 \tan^2 \frac{1}{2} (90 - \phi)$. The experimental value for ϕ of the material used is about 37°. It is probable, however, that both the density and the coefficient of resistance of the fill are changed.

The present method used in practice to take account of static surcharges assumes that ϕ is not changed, but that the density is changed. It requires that the readings for a concentrated load at any point on the wedge be the same. The test does not show this to be the case. It was impossible to place the load closer than 1 ft. to the wall. The results clearly show that the effect of a load depends on its distance from the wall.

XV.—Effect of Moving Loads on the Lateral Pressure of Horizontal Fills Against a Vertical Wall.

After completing the previous test, an attempt was made to measure the effect of loads rolling across the fill, parallel to the wall. The keg of nails was rolled three times across the width of the fill, at various distances

from the wall, starting at the greatest distance. Readings were taken directly after this rolling, the load having been removed from the fill. The previous test had compacted the fill fairly well, so that the effect was that of a moving load on a settled or tamped fill.

TABLE 31.—TEST No. 23.

Date: May 6, 1921.
Weather: Clear.
Temperature: 15° cent. (59° Fahr.)

Physical Properties: Same as in Test No. 1.
d = distance of surcharged load of 100 lb. from back of wall.

Readings are the averages of five positions of load.

Distance, d, in feet.	Pressures per Foot, in Pounds.			Tan ϕ' $= \frac{V}{H}$	$\frac{x}{h}$	Percentage Changes.				
	H.	V.	P.			H.	V.	P.	$\frac{V}{H}$	$\frac{x}{h}$
No load	469.2	286.8	551.6	0.610	0.367	.....	.....	.....	.....	.....
8.5	472.5	288.6	553.6	0.610	0.367	+0.7	+0.6	+0.4	0	0
7.5	473.4	288.9	554.6	0.611	0.367	0.9	0.7	0.5	+0.2	0
6.5	474.4	289.4	555.8	0.611	0.367	1.1	0.9	0.8	0.2	0
5.5	475.3	290.0	556.8	0.611	0.368	1.3	1.1	0.9	0.2	0.3
4.5	476.3	290.5	557.9	0.611	0.367	1.5	1.3	1.1	0.2	0
3.5	478.1	291.7	560.1	0.611	0.368	1.9	1.7	1.5	0.2	0.3
2.5	479.7	293.8	562.7	0.612	0.368	2.2	2.4	2.0	0.2	0.3
1.5	483.1	297.5	567.6	0.617	0.380	3.0	3.7	2.9	0.3	3.5
After test	483.5	299.3	586.6	0.620	0.380	3.0	4.5	3.1	1.6	3.5

Rolling or moving loads on a tamped fill have no effect on the pressure. In order to investigate the action of this compressed fill as it aged, it was allowed to stand for seven days. The summary given in Table 33 of pressures exerted by this fill is of interest. The height of fill was 6 ft., originally, $y = 100$ lb. per cu. ft., $\phi = 36°$.

The fill returns almost to its original state, except that the resultant rises 6.8 per cent. The decrease in the vertical component is due to the drying out of the sand along the wall. The maximum at 96 hours is due to a slight rupture in the fill. It is interesting to note that this packing and loading of the fill caused a variation of only 8.9% in the horizontal component, which is less than the variation in a similar fill caused by a change of 13° cent. in temperature over a like period of 166 hours.

XVI.—Effect of Static Loads on the Lateral Pressure of Horizontal Fill Against a Wall with Positive Back-Batter of 1:4.

A test similar to that in Section XIV was made with a 4-ft. fill and a wall sloping at + 1:4. The distance of the load is measured from the top of the wall. The heel of the wall was 1 ft. closer to the back of the bin, so that, in the last position, the load was directly above the heel of the wall.

The results of this test are similar to those obtained with a vertical wall. The change in vertical component disappears soon after the load is removed, whereas the horizontal component remains undiminished. In the last position, the 100-lb. load is directly over the wall, yet the increase in total vertical component is only 17 lb.

TABLE 32.—Test No. 24.

The pressures given are per foot of width of wall exerted by a 6-ft. horizontal fill against a vertical wooden wall, after a load of 100 lb. has been rolled across the fill at a distance, *d*, from the wall.

Date: May 6, 1921. Physical Properties: Same as for Test No. 23.

Distance, *d*, in feet.	Pressures, per Foot, in Pounds.			$\text{Tan } \phi' = \frac{V}{H}$	$\frac{x}{h}$	Percentage Changes.		
	H.	*V.*	*P.*			*H.*	*V.*	*P.*
No load	483.6	299.4	568.6	0.620	0.369	....	....	....
8.5	484.4	299.8	569.6	0.620	0.368	+0.2	+0.1	+0.2
7.5	484.6	300.0	570.0	0.619	0.368	0.2	0.2	0.2
6.5	484.6	300.1	570.0	0.620	0.368	0.2	0.2	0.3
5.5	485.0	300.1	570.4	0.619	0.369	0.3	0.2	0.4
4.5	485.4	300.4	570.8	0.619	0.369	0.4	0.3	0.5
3.5	485.8	300.6	571.2	0.619	0.369	0.5	0.4	0.5
2.5	485.9	300.9	571.4	0.619	0.369	0.5	0.5	0.7
1.5	486.6	301.6	572.4	0.619	0.370	0.6	0.7	1.0

TABLE 33.

	Pressures, per Foot, in Pounds.			$\text{Tan } \phi' = \frac{V}{H}$	$\frac{x}{h}$	Percentage Changes Over Original Values.			
	H.	*V.*	*P.*			*H.*	*V.*	*P.*	$\frac{x}{h}$
Just after filling.....	458.4	288.6	541.6	0.652	0.367	.....	.....	.....	.....
48 hours later.....	469.2	288.8	551.6	0.611	0.367	+2.4	−0.6	+1.8	0
After static test.....	483.5	299.3	568.5	0.620	0.380	+5.5	+3.7	+5.0	+3.5
1 hour later.....	483.6	299.4	568.6	0.620	0.369	+5.5	+3.7	+5.0	+0.5
After dynamic test.....	486.6	301.6	572.4	0.619	0.370	+6.2	+4.5	+5.7	+0.8
96 hours later (144 total).	499.0	287.0	575.6	0.575	0.384	+8.9	−0.1	+6.3	+4.6
118 " " (166 ").	464.8	268.2	537.0	0.577	0.392	+1.4	−7.1	−0.8	+6.8

Maximum values occurred at 96 hours (total 144 hours.)
Minimum " " " 118 " (" 166 ")

General Conclusions on the Pressure of Sand Fills, Deduced from the Cincinnati Experiments.

1.—The fill does not act like a liquid. The transmission of pressure does not obey the Pascal law.

2.—The fill does not act like an elastic solid. Loads on the fill cause a change in physical properties as well as a deformation. These changes slowly disappear, but not according to any definite law.

3.—There is no sharply defined wedge of rupture, nor can any surface of rupture be detected on the surface of the fill. However, loads on the fill beyond the wedge of rupture have practically no effect on the lateral pressure. Loads on the wedge of rupture do not increase the pressure by as much as would be required by a well-defined and distinct wedge.

TABLE 34.—Test No. 25.

Date: December 4, 1921.
Weather: Clear
Temperature = 31½° cent. (88° Fahr.)

Physical Properties: Same as in Test No. 17.
Height of fill = 4 ft.
Wall batter = + 1 : 4 (14° 00′).
Load = 100 lb. per sq. ft.

d = distance, in feet, of load from top of wall.

Distance, d, in feet.	Total Pressures on Wall, in Pounds.		Tan $\phi' = \frac{V}{H}$	Percentage Increases	
	H.	V.		H.	V.
No load	823	746	0.907	0	0
8	823	746	0.907	0	0
7	823	748	0.909	0	+ 0.3
6	823	748	0.909	0	0.3
5	823	748	0.909	0	0.3
4	825	748	0.908	+ 0.2	0.3
3	828	748	0.905	0.6	0.3
2	831	756	0.910	1.0	1.3
1	840.5	761	0.907	2.1	2.0
No load	840	763	0.908	2.1	2.3
1 hour later	840	747	0.890	2.1	0.1

4.—The lateral earth pressure, for all types and kinds of wall and for all types of loading and fill, acts at an angle to the normal to the wall equal to the angle of static friction between the fill and the back of the wall. The height of application is above the $\frac{1}{3}$ point, but below the $\frac{4}{10}$ point of the height.

5.—For a vertical wall with horizontal fill, the value of the horizontal pressure is given by the formula, $E = \frac{1}{2} y h^2 \tan^2 \frac{1}{2} (90° - \phi)$, in which ϕ is the angle of internal resistance of the fill, corresponding to the static friction of the filling material on itself. This formula is a special case of both the Rankine and Coulomb theories. The value of the vertical component is easily determined from a consideration of Conclusion 4.

6.—For all general cases, the pressure is obtained closest by the general wedge theory

$$E = \tfrac{1}{2} y h^2 C; \qquad C = \left(\frac{\cos(\phi - \alpha)}{(n+1)\cos\alpha}\right)^2 \frac{1}{\cos(\phi' + \alpha)};$$

$$H = E \cos(\phi' + \alpha); \qquad n = \sqrt{\frac{\sin(\phi + \phi')\sin(\phi - \varepsilon)}{\cos(\phi' + \alpha)\cos(\alpha - \varepsilon)}}.$$

The wedge theory that disregards wall friction gives too high results except for a vertical wall, in which case it gives results closer to the experimental results than the general wedge theory.

7.—The general wedge theory gives low results for a vertical wall, and it is better to use the wedge theory, disregarding the wall friction,

$$H = \tfrac{1}{2} y h^2 J; \qquad J = \left(\frac{\cos\phi}{n+1}\right)^2$$

$$E = H \sec\phi'; \qquad n = \sqrt{\frac{\sin\phi \sin(\phi - \varepsilon)}{\cos\varepsilon}}.$$

8.—The experimental results are not in agreement with the Rankine theory, except as noted in Conclusion 5.

9.—The effect of settling is to increase the pressure exerted by the fill, the maximum being attained within 2 hours of placing the fill. The pressure then decreases to the minimum within 24 hours, after which the pressure oscillates toward equilibrium between the maximum and minimum thus found.

10.—The pressure is a direct function of the temperature.

11.—The pressure is only slightly affected by humidity, except that, in dry weather, evaporation of moisture from the fill along the wall, near the surface, causes a greater horizontal and a less vertical component.

12.—Surcharges, both static and dynamic, compress the fill and increase the resultant pressure. Such an increase disappears in time, usually in about seven days.

Résumé of Previous Experiments.

In the early literature on the subject of lateral earth pressure, little is said about experiments, although many of the contributors to the theoretical development probably made some tests. Belidor, in 1720, stated that "experiment leads him to assume a 45° prism of rupture".* The earliest description of an apparatus is that of Gadroy who, in 1745, tried to check Belidor's assumptions.† Although his experiments in a bin 3 in. high showed a plane of rupture at a slope of 2 : 3 for a sand with a natural slope of 1 : 1, which is an experimental proof that the plane of rupture is not the plane of repose, the only conclusion drawn was that a wedge of rupture, moving on the plane of slope, did exist. The early method of testing was to construct a box with one side closed by a gate, hinged at the bottom. The top of the gate was restrained by a cord, passing over a fixed pulley, tied to a pan. Weights placed in the pan balanced the overturning effect of the pressure on the gate. It is, therefore, not surprising that a vertical component was not detected. It is surprising how many conclusions the experimenters did not notice. For example, Gauthey, in 1785, using a wedge-shaped box with the measuring gate vertical, obtained data showing that the pressure is constant for angles of the wedge between 22½° and 67½°, the material being sand. This proves that the wedge of rupture does not include all the material above the plane of slope, and yet, although Coulomb had issued his theory of the wedge of maximum pressure in 1773, Gauthey starts his theoretical discussion with the assumption of a wedge sliding on the plane of slope.‡

The danger of obtaining the passive pressure and the method of overbalancing the gate to insure that the active pressure is being determined, was first discussed by Woltmann in 1791. With a test gate 4 ft. square his results were much lower than those required by theory. The discrepancy would have been accounted for if he had taken into account the effect of a vertical component.§ Experiments by Mayniel in 1806 and by de Koeszegh in 1828 showed similar results.

As early as 1800, Coulomb's contributions were misinterpreted. Hagen comments on this, emphasizing that Coulomb had called the friction coeffi-

* Belidor, "La Science des Ingénieurs", 1st edition, 1729.
† Mayniel, "Poussee des Terres", 1808.
‡ Gauthey, "Ouvres", Vol. I, edited by Navier, 1809.
§ Woltmann, "Hydraulische Arkitektur", Vol. 3 and 4, 1794–99.

cient of earth, $\frac{1}{n}$, and had made no mention of any relationship between this coefficient and the natural slope. It was Woltmann who first made the assumption that the coefficient of internal friction was the tangent of the angle of repose. Unfortunately, he used this assumption in his German translation of Coulomb's "Essai", and it is from this translation that later writers copied the Coulomb theory.*

Hagen repeated Woltmann's experiments, and discovered that if he assumed a resultant inclined at a constant angle to the wall, there was perfect agreement between theory and experiment. The full-scale walls built and tested by Lt. Hope in 1845 showed results similar to those obtained from the Cincinnati tests.† A wall, 23 in. thick, failed under the pressure of 10 ft. of fill, the material being ballast weighing 95 lb. per cu. ft., and having a natural slope of 37°. The stability moment of the wall is 1 920 ft-lb. and the overturning moment of the fill is 3 970 ft-lb., disregarding the vertical component, as was done by the late Sir Benjamin Baker,‡ Hon. M. Am. Soc. C. E., in his discussion of Hope's tests. If the vertical component is taken into account, the overturning moment of the fill is 2 240 ft-lb. Baker's contention that the wedge theory gives a factor of safety of 2 (*i. e.*, 3 970 : 1 920), is not true, the overturning moment is slightly greater than the resisting moment.

Gen. Burgoyne, in 1834, built four 20-ft. walls, the average width of each being one-sixth the height, and tested them to destruction. No attempt was made to compare the results with theory; the application of the Cincinnati results gives the following:

Wall A: Both faces sloping 1 : 5, held a 20-ft. fill weighing 87 lb. per cu. ft. during heavy rains. M_r : 34 200; M_0 : 14 900.

Wall B: Vertical back, front battered at 1 : 5, held a 20-ft. fill after tipping about 2½ in.; M_r : 32 400; M_0 : 17 350.

Wall C: Vertical front, back battered at 1 : 5, failed by bulging and bursting at one-third the height when the fill reached a height of 17 ft. There was no tendency to overturn. M_r : 17 000; M_0 : 6 000; failure due to low shear strength.

Wall D: Both faces vertical, failed by overturning when the fill reached a height of 17 ft.; M_r : 15 500; M_0 : 17 500.

The result of each test agrees closely with the Cincinnati results. Because the fill was saturated, it was assumed that the unit weight was 100 lb. per cu. ft., the natural repose, 34°, and that the resultant acted at an angle of 34° to the normal, the walls being of rough rubble, laid dry.§

Aude, in 1848, obtained values that were about 85% of those obtained by the Coulomb theory.‖ Considère, in 1870, showed that the results agreed very closely with theory, by taking into account the wall friction.¶

* Hagen, *Annalen der Physik und Chemie*, Vol. 116 (1833), pp. 17–48, 297–323; *Berichte* der k. Akademie der Wissenschaften (1853), pp. 35–42.

† Hope, London Royal Engineers Papers, Vol. 7 (1845), pp. 69–86.

‡ Baker, "Lateral Pressure of Earthwork," 1883.

§ Owen, *Transactions*, Irish Inst. Civ. Engrs. (1845), Vol. I, pp. 138–144.

‖ Aude, Memorial de l'Officier du Génie, Vol. 9.

¶ Considère, *Annales des Ponts et Chaussées* (1870), Vol. 19, p. 547.

Winkler, in 1863, found a resultant always inclined at an angle equal to the angle of friction on the wall. To correct his results for the side-wall resistances, he repeated the tests with a thin partition wall in the center of the bin. From the difference in readings, he obtains the retarding effect of a side-wall. This method would be reliable if the width of the bin was so great that there would be no chance of arching action in the narrow bins. By the use of two axes of rotation, one under the inner face of the wall and the other at the toe, he obtains the vertical and horizontal components, because in the first reading the vertical component is eliminated and the second shows the effect of both components.* The objection to this method is that, in order to determine the total pressure, two independent tests must be made, and no account can be taken of changes in the conditions, which must be identical for accuracy. The experiments of Constable in 1874† and of Darwin in 1877‡ were made on so small a scale that the results are unreliable.

Leygue, in 1885, reported a very complete set of experiments covering every phase of the earth-pressure problem. He proved experimentally the co-existence of friction and cohesion in soils, an assumption made by Coulomb, but not accepted by some of the later scientists. By the use of a bin with glass walls, he investigated the shape of the wedge of rupture. He discovered that the surface of rupture varied but little from a plane, and that its shape was independent of the height of the fill, the method of wall failure (rotation or translation), of the nature or the slope of the wall, and of the surface slope of the fill. His quantitative results are not so valuable because of the small sized apparatus. Winkler's method was used to determine the effect of the side-walls.§

A large apparatus was constructed at the University of Nebraska by A. A. Steel in 1899, a test wall 14½ ft. high being inserted in a pit. The horizontal and vertical components on two 12-in. square sections of the wall were measured by levers. For such a depth the pit was too small in cross-section, and the effect of the solidified earth walls caused inconsistent readings. The labor involved prohibited more than a few tests and some of these were performed in parts. Steel noticed the drop in pressure over night, which he attributed to interference with the apparatus by troublesome boys. The same drop in pressure was noticed in the Cincinnati tests, during which the apparatus was safe from intrusion.‖

The Mueller-Breslau apparatus, on which the Cincinnati design was based, was "an attempt to reproduce the actual action of a retaining wall under earth pressure". A complete translation of the section of Mueller-Breslau's book dealing with his apparatus and experiments is included in the writer's thesis, filed in the Library of the University of Cincinnati. Mueller-Breslau determined the horizontal and vertical components by measuring the deforma-

* Winkler, *Der Civilingenieur* (1865), Vol. II, pp. 1–11; "Neue Theorie des Erddrucks" (1872). (Not obtainable).

† Constable, *Transactions*, Am. Soc. C. E. (1874), Vol. 3, pp. 67–75.

‡ Darwin, *Minutes of Proceedings*, Inst. C. E. (1883), Vol. 71, p. 350.

§ Leygue, *Annales des Ponts et Chaussées* (1885), Vol. 10, pp. 780–1000.

‖ Steel, *Engineering News* (1899), Vol. 42, pp. 261–263.

tions in calibrated steel struts, restraining a wall 29 in. high. No attempt was made to measure the effect of the side-walls, which were coated with graphite to reduce the friction. One noteworthy observation, which caused the use of graphite, was that the angle of friction of sand on these walls was 14°, whereas that of sand on glass was 22°.*

His conclusions were that the measured earth pressure is somewhat greater than the value from the Coulomb theory. By using the angle of internal resistance in place of the angle of repose, the writer has shown that very close agreement exists between Mueller-Breslau's results and the wedge theory. The surface slope had no effect on the inclination of the resultant. Using glass and emery-cloth backing on the wall, he concluded that the resultant is always inclined at the angle of friction on the wall. Alternate loading and unloading of the fill caused the resultant to rise. The same results were obtained in the Cincinnati experiments.

A photographic investigation of the wedge of rupture was made in a glass box about 2 ft. high. The wall could be displaced by either a translation or a rotation. It was found that a movement of 1 to 6 mm., had practically no effect on the shape of the surface of rupture, which was slightly curved down at the lower end, approximating the Coulomb theory very closely. With surcharged fills, the lower end of the surface of rupture was straight and the upper portion is curved. Mueller-Breslau announces that he has designed a much larger apparatus, but the writer has been unable to find any further information than that given in his book of 1906.

At the University of Pittsburgh, Professor J. H. Smith constructed an apparatus for the determination of the lateral pressure. As the maximum distance between the front and the back walls of the box he used was only 18 in., the height being 5 ft., the results are only applicable to storage bins. The use of these results to retaining-wall design is equivalent to the assumption that earth pressure is the same as liquid pressure.†

Crosthwaite (1916-22) investigated the accuracy of Rankine's formula for penetration, $d = \frac{P}{y}\left(\frac{1 - \sin \phi}{1 + \sin \phi}\right)^2 = \frac{P}{y} \tan^4 \frac{1}{2}(90 - \phi)$, by using a plunger of $3\frac{1}{4}$ sq. in. bearing area. Substituting the penetrations found,‡ the value of ϕ can be computed. Comparison is then made between this ϕ and the natural slope. Sand, garden earth, cinders, ashes, and clay were used. It was found, in general, that, in materials loosely deposited, the angle of friction equals the angle of repose. On consolidation of the fill, this did not hold true. The results obtained are the combined effects of cohesion and friction; there is no necessity to consider these phenomena separately. Although Crosthwaite draws definite conclusions, it should be noted that, in most of his experimental work the results were widely scattered, many readings being 50% from the mean. He considers this to be due to the small-sized apparatus, and states, "Model experiments are best. * * * of the experiments made by previous investigators to investigate the lateral pressure of earth, those in which model

* Mueller-Breslau, "Erddruck auf Stützmauern", 1906.

† Smith, *Proceedings*, Am. Soc. for Testing Materials, Vol. 15, Pt. 2, pp. 382–398.

‡ Crosthwaite, *Minutes of Proceedings*, Inst. C. E., 1916, Vol. 103, and 1920.

walls were used are of the greatest value, but if the models are of any size, the experimental difficulties are almost insuperable." The writer agrees with this last phrase.

Experiments with sand ranged from dry sand to saturated sand (35% moisture), with a range in weight from 108 to 121 lb. per cu. ft. The computed value of $\frac{1 - \sin \phi}{1 + \sin \phi}$ was 0.0500 to 0.0601, corresponding to values of $\phi = 64° 27'$ to $62° 27'$. The sand acted like a solid, the plunger came to rest immediately. The observed natural slope was 52° 28′.

Garden earth, with a natural slope of 46° 12′, gave experimental values for ϕ of 43° 08′ to 60° 51′, with an average value of 58° 15′. Ashes and cinders acted as viscous solids, and required a long time to come to rest. The results obtained are summarized in Table 35.

TABLE 35.

Materials.	Angle of repose.	Maximum experimental value of ϕ.
Leighton Buzzard sand, damp	52° 28′	67° 48′
" " " , dry	48° 04′	64° 47′
Bournemouth sand	47° 21′	65° 32′
Garden earth	46° 12′	60° 51′
Ashes and cinders through ¼-in. sieve	53° 48′	72° 59′
" " " 30 by 30 mesh	47° 30′	68° 08′

Experiments with clay were made, with a bearing area of 1 sq. ft., and loads up to 12 tons per sq. ft. With mud, the area was 16 sq. ft., with loads up to 1.25 tons per sq. ft. These tests were made in the natural undisturbed ground, and showed that the internal coefficient of friction decreases rapidly with an increase in pressure. There were great variations in the results.

Crosthwaite concluded that the Rankine formula holds true, but that ϕ must be taken as the angle of internal friction and not as the surface slope. For clay, the penetration varies as the square of the load. Had the experimenter taken the trouble to determine the actual coefficient of friction in the material, he would have found that the angle of internal friction is always less than the natural slope, whereas, in these tests, the average angle of friction is 68° 21′, with a corresponding natural slope of 48° 24′, almost 20° more. Without doubt, the effect of friction along the sides of the bucket used as the container had considerable influence on these results.

In his 1920 report, Crosthwaite gives the opposite conclusion for the same type of experiments: "The angles of internal friction from these experiments was much less than the angle of repose, and was not constant for any one material, but depended on its state of aggregation, whether it was put loosely together, shaken, or consolidated by tamping."

Tests with a small vertical wall, hinged at the base, in which the overturning moment caused by sand fills was measured by the tension in a string tied to the top of the wall, caused Crosthwaite to conclude that the pressures

as calculated from the Coulomb and the Rankine formulas are too high, especially for surcharges. Walls calculated by the Rankine theory, using the value of the angle of repose for ϕ, would have a factor of safety of $2\frac{1}{2}$ to 4.

In order to test the wedge theory, a false bottom coated with glued-on sand was inserted, that could be set at any angle with the back of the wall, so that the pressure produced by any wedge could be measured. He found that the wedge theories that take into account the friction between the wall and the fill, give correct results for the wedge of maximum thrust as long as the wall is not surcharged, but that the calculated pressures are 30% too great for surcharged walls. These results, he states, made him think that the theory was wrong, that the friction between the wall and the fill should not be considered, and that ϕ was really the angle of friction. In view of the result just stated, that the theoretical and experimental results did agree, it is difficult to understand how Crosthwaite obtained his conclusions. He calculated the value of ϕ required to satisfy the values obtained for the lateral pressure of a horizontal fill against a vertical wall. This value, 42° 20′, he called the angle of internal resistance. The natural slope of this material was 35°, or less than the angle of friction. With this new value for ϕ and disregarding the friction on the wall, "the maximum calculated pressures were practically identical with the observed."

The next experiment was intended to determine whether or not wall friction did affect the horizontal pressures. A long box was sawed, at the middle of its length, into two compartments by a vertical cut. One-half the box was fixed to solid supports and the other half was placed on rollers. When the box is filled with sand, the two compartments are urged apart by the pressure of the sand on the plane passing through the saw-cut. By measuring the tension in a string that prevented the half of the box on rollers from being pushed forward, the pressure on the vertical plane was determined. The part of the box on rollers was then turned end for end, so that the sand in the fixed compartment was held in position by the solid end of the box, the face of which was coated with glued-on sand. "This is the same as if it had been retained by a rough wall, and if the friction between the wall and backing affects the amount of the horizontal thrust the amount of tension now required to keep the rolling box in position should have been less than before." A large number of experiments showed that there was practically no difference, and, therefore, Crosthwaite concludes that "it is not correct to assume that the friction between the wall and backing can affect the amount of the horizontal thrust."

Disregarding the truth of the conclusion, the writer doubts the right to draw such conclusions from the tests. What was shown was that the thrust of a mass of sand on a theoretical vertical plane bounding the mass is the same as that on a vertical wall replacing the material on the other side of the theoretical plane, if the surface of the wall is of the same material as the filling material. Had the experimenter used a smooth wall and obtained the same results, the conclusions might have an experimental foundation.

The final conclusions drawn by Crosthwaite from all his work are:

1.—That the plane of rupture may be a convenient mathematical assumption, but has no existence in the granular material dealt with—at least, he was unable to trace any evidence of it in his experiments.

2.—That the angle of repose is a physical constant that relates only to the surface, and is represented in the interior of the mass of sand by the angle of internal friction.

3.—That the angle of internal friction is not a physical characteristic constant for any one material, but varies with the state of its aggregation.

4.—That friction between the back of a wall and its backing does not affect the amount of the resultant thrust. The writer believes that "horizontal" component of the thrust is meant, because the apparatus used gives no measurement of the vertical component.

5.—That the wedge theory, which takes into account the wall friction and the angle of repose, although giving correct results when applied to a wall without surcharge, or with negative surcharge, breaks down completely when applied to a surcharged wall.

The writer desires to call attention to the statements concerning the relative value of the angles of repose and internal friction. Although the articles in *Engineering and Contracting* for March 31, 1920, and *Engineering News-Record* for January 19, 1922, begin with a statement that "the angle of internal friction from these experiments was much less than the angle of repose," all the data in these and other articles by Crosthwaite show the opposite. The other conclusions are remarkably close to those drawn from the Cincinnati experiments, although the experimenter's right to give them from the tests he performed is doubtful.

By using a rotating test-wall apparatus in a bin 7 ft. high, Fulton,* in 1920, measured the overturning moment of various types of fill. The magnitude of the resulting moment was measured by observing the extension of a bar fitted with a Ewing extensometer.

The results were compared with the various theories, and, using the conclusions given in the abstract, it was found by Fulton that:

1.—The overturning moment on retaining walls due to the filling, as calculated from a general formula based on the wedge theory, is approximately the same as that which ordinarily exists when the filling is within the limits of these experiments, and when the inclination of the inner face of the wall either outward or inward is not greater than usually obtains in practice. In calculating theoretical values, it was assumed that the resultant acts on the inner face at a point one-third of the height of the fill from the bottom of the wall and that it makes an angle equal to the angle of repose with the normal to the wall.

2.—That in walls having a filling the surcharge of which is not continuous and unlimited, the general formula requires modification. This modification can be made by reducing the sliding prism of irregular section to one equivalent triangular section from which the overturning moment can be deduced either by direct calculation or by a convenient graphical method. Under these

* *Minutes of Proceedings*, Inst. C. E., 1920. Abstract in *Engineering and Contracting*, March 31, 1920.

conditions, the position of the resultant pressure may range from 0.33 to 0.364 of the height of the fill from the base of the wall.

For a vertical wall with a horizontal fill, the Rankine theory gave results for the overturning moment greatly in excess of the observed values, whereas the wedge theory gave results quite close to those obtained by experiment. For surcharged fills and a vertical wall, the Rankine theory gave values 50% in excess, and the wedge theory 20% in excess, of the observed values. Although three materials were used, clean river sand, gravel, and garden soil, it seems that most of the tests were made with sand. In order to keep a full plane of rupture inside the fill, surcharged fills were made smaller than the height of the bin, 7 ft., hence it may be concluded that the bin was rather narrow. With sloped walls the theoretical values were from 40 to 50% in excess of the experimental values.

Moulton* classifies all materials between liquids and solids, as:

A.—Solid, from moist sand to rock, dealing with walls.

B.—Granular, from dry sand to wheat, dealing with bins.

C.—Semi-liquid, from wet clay to quicksand.

He dealt only with the first and, from many field observations, concluded that the surface of rupture and surface of repose are not plane; that fracture in these materials is conchoidal. He notes that in one of the subway tunnels, the floor of which was 68 ft. below street level, a break in the surface occurred at a distance of 50 ft. from the center line, the average line of rupture being on a slope of about 1 : 2. In the placer mine gravel banks at Dawson, Yukon Territory, Canada, the natural slopes averaged about 1 : 2, for heights up to 100 ft., and were always concave. He concludes, therefore, that the wedge behind a wall must exert pressure in accordance with Meem's theory.

It may be noticed that the writer makes no comment on the experimental work of E. P. Goodrich and J. C. Meem, Members, Am. Soc. C. E., and the discussions thereon, as published in the *Transactions* of the Society. Some recent work by Finnan and Hummel, of the Institution of Civil Engineers (1921), in general agrees with the writer's results. As the work is still in progress, no more than mention is required. Terzaghi† rejects all the existing theories and starts the investigation of the stresses in granular fills, following the method of Couplet (1727), but including the frictional forces between grains. The lateral pressure per unit area due to liquids is given by the well-known hydrostatic formula, $p = wh$. The same law holds true for a mass of smooth spheres of equal size. For any other assemblage of smooth spheres, the lateral pressure may be greater or less and is denoted by $k\,w\,h = p'$. Statical resistance is called the cause of the difference between p and p', because it exists independently of any frictional resistances acting at the surface of the spheres. The statical resistance is merely the effect of the weight of individual grains transmitted by their neighbors to the wall of the confining vessels.

Should the spheres become rough, the value of p' is unchanged as long as the wall does not yield. After a given yield, frictional resistance will be set

* *Transactions*, Am. Inst. of Min. and Metallurgical Engrs., 1920.

† *Engineering News-Record*, September 30, 1920.

up, due to the roughness of the grains requiring that work be done in deforming the shape of the body. During the failure of a wall, one may distinguish between the first phase, the interval in which the frictional resistance is increasing, but has not yet reached a maximum, and in which the statical resistance remains constant, and the second phase, during which the mutual positions of the grains change. In a detailed study, the elastic properties of the grains must also be considered.

The experimental work, described in *Engineering News-Record*, of September 30, 1920, consisted of several sets of tests, the object of each of which was to investigate one single point in the theory. Although several diagrams are given, no scale or dimensions is given. At the time this article appeared, the writer was working on his apparatus and, therefore, was interested in knowing the type of apparatus and method used by Dr. Terzaghi. In reply to a letter, Dr. Terzaghi states in part:

"The preliminary investigations made for my own experiments have shown that the intensity of the earth pressure is strongly influenced by the side of the bin, unless the width of the bin is at least two and a half times greater than the height of the back-filling. As shown by Donath (1891) the use of scales to measure the vertical component is utterly unreliable because of the inevitable vertical displacement of the wall. That is the reason why I adopted an indirect method for measuring the vertical component [several comments are here inserted on the design of the Cincinnati apparatus].

"My own experiments were made on a very small scale, the height of the retaining wall being 10 cm. (4 in.) only. I was not interested in the quantities but in the principle and in this case the size of the box makes no difference. The goal of my investigations was the determination of the elastic constants of the sand."

Keeping in mind the size of Dr. Terzaghi's apparatus, the investigation covered the following problems:

1.—Earth Pressure at Rest.—The ratio of lateral to vertical pressure was determined. The vertical pressure was applied by a testing machine on a mass of sand in a square frame (no dimensions given), one side of which was closed by a steel tape. Pressure on the sand caused this tape to press against a similar tape. A third tape was passed between these two and by pulling this third tape out, the frictional force on it was measured. No account is taken of the resistances of the sides of the frame, nor of the pressure lost in deflecting the inner tape before it began to bind on the measuring tape, although the distance between the tapes could be made very small.

2.—Horizontal Component.—The test wall (10 cm. high) rested on frictionless rollers, and was tied to a weighing scale with a cord passing through the fill. The connection to the wall was probably at the third point. Displacement of the wall was horizontal (sliding) and occurred by reducing the weights on the scale so that the active pressure was obtained. Coarse sand (2–3mm.) was used, because it is easier to produce a fairly homogeneous back-filling with a coarse sand than with a fine sand. In order to obtain an idea of the size of the apparatus, note that 40 grains of sand in a line will extend the entire height of the wall.

3.—Vertical Component.—By using a single roller under the wall, and using an indirect method, Dr. Terzaghi claims to measure the vertical component with only a horizontal displacement. The bottom of the front end of the wall is provided with a steel button resting on a ground-glass surface. The filling exerts a vertical component on the wall causing this steel button (placed very close to the bin) to bind on the glass. Weights are placed on the farther end of the base of the wall until the wall is balanced over the roller, which is at about the mid-point of the base. The computation of the vertical component requires the use of the horizontal component which was independently obtained. Dr. Terzaghi states "no pull is exerted in a vertical direction by the measuring device on the retaining wall". He neglects the process of counterbalancing which causes the face of the wall to move up a trifling distance, it is true, yet a vertical displacement does occur. The writer wishes to note that in all present-day methods, stress is determined by strain in the direction of the stress; in other words, it is impossible to measure a vertical force unless there is a vertical displacement.

4.—Passive Pressures.—Passive pressures were obtained in the same manner as the active components, but measurements were made while the wall was being forced against the fill. As shown in the Cincinnati experiments, the amount of such movement must be considerable, to correspond to the amount of yield in the "second phase" (as termed by Dr. Terzaghi), before the passive pressure will occur.

5.—Observing Inter-Granular Movements.—Sheets of aluminum were cut in the shape of sand-grain sections and a study was made of the manner in which these sheets were assembled. Various other tests, measuring the lateral pressure at rest and during the second phase, are given. During the first phase the angle of friction on the wall being 34°, the resultant acted at angles of from 5 to 20° for various kinds of fill. The measured angle of inclination increased with the total pressure. During the second phase, the lateral pressure, for all cases, acts normal to the back of the wall.

In discussing the action of the sand during passive pressure, Dr. Terzaghi also notes two phases. During the first phase, the resistance of the back-filling increases in direct proportion to the distance of compression, until the frictional resistance which acts between the surface of contact of the sand grains assumes its maximum value. The second phase starts with inter-granular movements, the intensity of which increases more rapidly than the earth pressure decreases. These movements are confined to a definite wedge of the mass resting against the wall, which statement can be obtained by a study of the equilibrium of a mass of spheres. This "slip" occurs only if the angle between the forces acting on some surface (surface of rupture) and the normal exceed a certain value, which is the "angle of slip".

"The earth pressure against a perfectly rigid wall is fairly independent of the back-filling." For sand, its value is $\frac{1}{2}$ (0.42) $y\,h^2$, and the minimum value for any back-filling can be caused by a yielding of the wall, and the corresponding coefficient varies from 0.15 to 0.05. It should be noted that all these formulas include the density, in spite of the previous statement that the

density is independent of the material. The conclusions are given for the sake of completeness, not because of their quantitative value. Dr. Terzaghi's work furnishes many valuable openings for future investigation.

The writer wishes to call attention to the close agreement of the independent experimental work, when such work was on a reasonably large scale and resulted in consistent conclusions, with the results of the Cincinnati tests. He believes that the smallest sized apparatus for the accurate determination of lateral earth pressure should be at least 5 ft. high, as was recommended by Professor Cain, in 1915. Fairly accurate readings can be obtained with a height of 3 to 4 ft.; a test wall 6 ft. high will give at least three accurate readings 1 ft. apart, from which reliable results can be obtained. The readings should be taken as directly as possible, friction dynamometers, springs, elastic gauges, etc., require too much calibration. Platform scales have proved reliable, accurate, and sensitive. Experiments on miniature apparatus are of no use; no one claims quantitative results, and the qualitative results are not consistent with each other.

Greater attention must be paid to the properties of the material used in the tests. Physical, as well as chemical and geological, analysis, including the determination of all the properties of the soil, is necessary for any comparison with the work of others.

Summary

From the Cincinnati experiments, together with the results of other experiments, the writer recommends the following rules for the determination of the lateral active earth pressure, to be used in the design of retaining walls and similar structures:

1.—The resultant pressure is inclined to the wall, deviating from the normal by an angle equal to the angle of friction between the fill and the wall.

2.—The resultant acts above the third point, and, for a heavy surcharge, as high as the 0.4 point.

3.—The horizontal component is given closest by the wedge theory, taking as the angle, ϕ, the experimentally determined angle of internal resistance of the fill.

The effect of surcharge, temperature, settling, etc., is found at the end of the report of the Cincinnati tests. The writer hopes to see the work at the University of Cincinnati continued, thereby completing the solution of the accurate experimental determination of the lateral pressure of granular materials.

DISCUSSION

J. C. MEEM,* M. AM. SOC. C. E.—It would be impossible to speak except in the highest commendation of this master effort, combining as it does research, compilation, analysis, and deduction, and the speaker wishes to express his high appreciation of the author's work, and to state his belief that the publication of this paper indicates that great progress has been made in the solution of problems pertaining to lateral earth pressure. The speaker feels convinced that although he differs from the author—and at times perhaps radically—such differences may not be irreconcilable.

His one and only criticism of the paper and its findings, therefore, is intended to be constructive and to involve a fundamental point, *viz.*, that the testing box or apparatus was not of sufficient width in relation to its height; that is, in order to get results wholly unaffected by lateral arching, horizontal as well as vertical, the box should have been much longer, by at least three or four times the height. That there is such arching action is as uncontrovertible as the existence of lateral pressure itself, on which it must depend; and there is reason to believe that it exerts itself horizontally, or nearly so, as well as vertically, with certain modifications. The speaker is convinced that the size of the testing apparatus is not as important as its relative proportions. For example: If a box, Fig. 7, is built, having a width, D, equal to its length, L, and its height, H, and if a false bottom, B-C, is put in position, as shown, and the box is filled with sand, the bottom may be supported by bolts through the sand and through a smaller top or washer bearing on the sand. The bolts may be tightened so that the box may be carried about without dislodging the sand. The support of this sand is entirely due to the transmission of its lateral pressure to the sides of the box, and it must undoubtedly be true that there is equal pressure on each of the four sides of the box. It is seen, therefore, that the measurement of the pressure on any one side is not an accurate determination of the lateral pressure, as would be the case if the length of the box, L, was increased until it very greatly exceeded its height, H. Any measurements should include the whole area, as nothing can be gained by measuring the pressures of fractional areas separately.

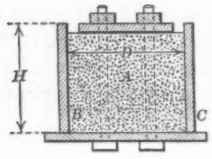


FIG. 7.

In Fig. 7 it can be shown that more satisfactory results can be obtained in a box 12 in. wide, 12 in. deep, and 4 ft. long, than in any cubical box of however large a cube.

If, in Fig. 7, it is assumed that the pressure of all the sand or soil above B-A-C is transmitted laterally to the sides, it does not matter whether the measurements of B-A-C, or of H and D, are in terms of inches, feet, yards, or rods, for if the testing material is homogeneous and the length is always sufficient to be greater than 4 H, the relative areas of soil-producing pressures may always be encompassed by the same geometric

* Vice-Pres. and Chf. Engr., Frederick L. Cranford, Inc., Brooklyn, N. Y.

figures. For instance, if the area producing pressure against the sides is measured in terms of one-half of all the sand above *B-A-C*, it does not matter whether the area is expressed in inches, feet, or rods. For the casual reader, it should be noted that this does not mean that the pressure per area is the same in all cases, but that the pressure per area can always be measured by the same relative formula in terms of the height.

In the case of small models for testing, objection is often made to the use of perfectly dry sand; but the reason for this is apparent when it is realized that cohesion in small models is relatively a much greater factor than it is in large models; that is, a cubic foot of moist sand may be cut vertically or even moulded or tunneled, whereas a deep cut or a large tunel through sand of the same consistency would break, or tend to break, down. Thus, the dry sand gives more nearly the same relative pressure in terms of the height.

After all, it is not difficult to realize that the results obtained by small models with dry sand are applicable (with slight modifications for changes in factors of moisture and cohesion) to operations of any size, even including those of a tunnel through solid rock, when the width of the tunnel is so great that the rock tends to break or become a mass of aggregates equivalent to large boulders. In ordinary soil, with which nearly all engineers in the East are familiar, there is usually an overlay of clay or hard compacted soil for varying depths; below that, and usually within the prism of the operation, sand is often found. The overlay of compacted soil does not usually tend to break down until the excavating operators have disturbed, or caused movement of, the sand, and then there occurs the phenomenon, referred to subsequently by Mr. Haines, and elsewhere by Mr. Moulton, of an almost vertical crack, caused by a section of clay which is pulled away by its own weight and the slight lateral movement of the sand at its base; and although the crack may seem to be continuous and to separate absolutely the two masses of soil, there must undoubtedly be spawls, cobbles, etc., projecting here and there and acting as braces, which prevent the soil back of them from breaking down and filling the crack. Of course, this applies as well to all more or less cohesive soils which are deeply cut or tunneled, even though there is no stratum of sand.

The measurement of the lateral pressure of this mass against the sheathing or structure is equivalent to that of a boulder resting on a background of other boulders, the pressure of which on the side of a small model would be slightly different, of course, from the resultant pressure of a sliding aggregate of small particles, such as dry sand; but it should not be difficult to correlate the pressure by proper assumption as to a mean angle of repose, or, more correctly perhaps, to an angle of internal friction, as noted by the author, and probably dependent on the angle of repose.

Bridges and foundations—in fact, all structures—are designed for maximum loads or conditions which they are rarely if ever required to meet. Why, then, in the determination of soil pressure, should not a relatively safe angle of repose be assumed for all conditions, modified, if necessary, by the angle of internal friction or as determined. For almost all conditions of soft

ground, other data being unavailable, the speaker uses an angle of repose of 45°, modified by an angle of rupture half way between that and the vertical. This angle of rupture, thus far, has been only an assumption, and if Mr. Feld will show that an angle of internal friction should be used instead of it, and give and determine its properties more definitely, the speaker feels that this alone will be a long step in advance.

The speaker, at one time in his crude experimentation, constructed a box of 5 ft. cube with four movable boards at the front (Fig. 8), each of these boards being tied over a pulley to a pail holding water or sand. No special results were obtained, however, because of the collapse of the boards with the initial movement. Later, he constructed arms on which the boards might slide, but discontinued the experiment, realizing that the lateral arching, both horizontal and vertical, prevented accurate determinations. He believes, however, that if a box is built, 10 ft. wide, 6 ft. high, and 24 ft. long (to prevent lateral arching), and the front boards are attached to heavy full-length I-beams rolling on bearings on arms attached to the box (Fig. 9), and if these boards are held back by cords passing to spring balances or to adjustable weights, the results obtained will be reasonably conclusive as to the determination of lateral earth pressure.

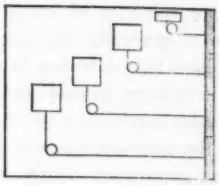


FIG. 8.

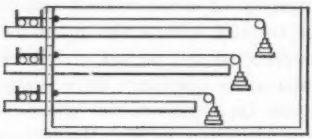


FIG. 9.

Again, it may be suggested that such a determination, even if made accurately and conclusively for dry sand, would not be applicable to soils of other conditions. The speaker, however, believes firmly that from such a determination the pressures with all kinds of soils can be formulated with reasonable accuracy.

Relative to this, the speaker believes that, in the ordinary or practical determination of soil pressures, five classifications of soil are all that are necessary, as follows:

(1) Cohesive.—Hard rock or clay, which does not break down or tend to break down under the operation.

(2) Semi-cohesive.—Normally dry sand, clay, gravel, etc., in fact, all soils ordinarily encountered which break or tend to break down under the operation.

(3) Non-cohesive.—Dry hot sand, such as beach sand, etc. (Dry grain may be included under this classification.)

(4) Semi-aqueous.—Water-bearing sand, gravel, etc., where the introduction of water does not affect the stability of the soil materially. (For instance, if a box is filled with dry sand reasonably free from finer particles such as clay, bog, etc., and if this sand is loaded, no radical change will be noted if water is introduced, except

that there may be some slight settlement due to the compacting of the soil and a change in its cohesive properties. If, however, an excess of clay, bog, or other so-called non-granular soil is incorporated with the sand, the introduction of water causes these particles to be displaced by pressure, almost as if the mass was pure clay or bog, and this gives a fifth class called aqueous.)

(5) Aqueous.—Plastic clays, quicksand, bog (also mercury, grout, pure water, etc.), soils which are displaced by ordinary pressure and, when confined under pressure, transmit that pressure in all directions equally.

It is interesting to note that plastic clay, containing as much as 40% of fine or medium granular sand, may be jacked into existing voids for considerable distances through specially constructed hydraulic rams.

The speaker believes it to be not only possible but probable that simple and practical formulas for the lateral pressure of earth will finally be established, modified only as required for each of the foregoing classes, but not by the necessity for determining separately the pressure of each type of soil, as clay, sand, gravel, etc., to analyze this.

Aqueous soils, Class (5), may be eliminated, because the determination of the pressures in such soils is simple, following the laws of hydrostatic pressure, with due regard, of course, to the fact that application should be made on the basis of the specific gravity of the basic liquid, and not the weight of the mass per volume.

Class (1), or wholly cohesive materials, such as solid rock or hard clay, can also be eliminated, as there is no pressure to be accounted for, and the only necessity for timbering or lining, if required, is to prevent disintegration or accident by spawling, etc. When the operation is large enough to cause the rock, clay, etc., to tend to break down, it automatically enters Class (2). and thus it is necessary to deal with only Classes (2), (3), and (4).

In Class (4), or semi-aqueous materials, in which the water is contained in or seeps through the voids without causing erosion, there are two distinct or different types of pressure acting in unison, first, the lateral pressure of the soil, modified only slightly by a change in its cohesive property, due to the presence of water, but otherwise conforming to the laws of lateral pressure of normally dry or semi-cohesive soils. The water exerts pressure through the voids of the soil, and finds necessarily reduced areas on which to exert its full head or pressure. Gauges, for instance, through the walls of submerged structures buried in soil, almost always show pressure due to the full hydrostatic head; and although it is true that these reduced areas have full-head water pressure, there are between them larger areas in which the soil pressure acts alone. If this was not true, the structure would be wholly surrounded by water pressure, and in all cases buoyant.

It should be noted that, when the cover is very shallow, or the operation calls for a constant churning of the material, it will always be safer to design for full or net hydrostatic pressure; that is, in all cases of sub-

merged structures with shallow cover, the maximum pressure should be considered, whether aqueous or combined. Analysis of Class (4), therefore, results as follows: The total pressure, as noted in Classes (2) or (3), is equal to the earth pressure plus the pressure of water over a reduced area due to the hydrostatic head. The speaker believes that 30%—or at most 40%—of the total area of water pressure is all that acts ordinarily on a submerged structure, but, for safety, he uses 50%, that is, P = total pressure; $P\,X$ = earth pressure; $P\,W$ = water pressure; therefore, $P = P\,X + \frac{P\,W}{2}$.

It should be noted, especially by the casual reader, that this determination applies only to static or approximately static conditions, and that where there is sufficient flow of water to cause erosion or collapse, hydraulic considerations must govern.

Finally, as to Classes (2) and (3), to which it is seen that the other classes may be directly or indirectly referred, it has already been noted that Class (3) should give the same formulas as Class (2), modified by the factor of cohesion, which changes the angle of repose more or less materially, but does not otherwise cause any changes in a formula applicable to either case.

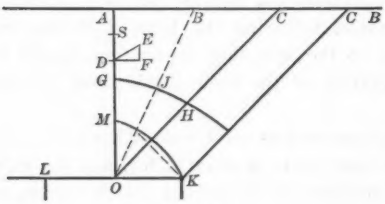


FIG. 10.

Assuming a trench, of depth, *A-O* (Fig. 10), which is sheeted and braced as it is excavated, continuous arching action is set up by virtue of the lateral pressure between the face, *A-O*, and the undisturbed ground, *O-C*, and when the full depth is reached, there is —to express it somewhat crudely —set up a sand arch, *A-G-H-C*, which undoubtedly possesses the three necessary fundamentals of a true arch, that is, lateral pressure at the key, *B-J*, thrust against the haunches, *A-G* and *H-C*, and internal cohesion supplied by the soil below *G-H*, which holds its so-called centering in place. The part of the area that causes pressure in *A-G* must, of course, be determined by experiment, but the speaker has assumed it as being the whole of the area above *G-H*, to the left of the plane, *B-O*, bisecting the angle, *A-O-C*, or $\frac{90° - \phi}{2} = 45° - \frac{\phi}{2}$, ϕ being the angle of repose.

He has also assumed that the thrust is due to the weight divided by the tangent of ϕ. The formula thus developed results with modifications as follows:

P = lateral pressure in any area;
A = Area (volume) of soil covering P;
W = Weight per volume of soil;
ϕ = Angle of repose; thus, $P = A \times \frac{W}{\tan \phi}$.

The center of pressure of the whole mass, *H-B-O*, is taken to be where a line or plane drawn through the center of gravity of the mass parallel to the angle of repose intersects the face of the sheeting or wall, and ordinarily should be found slightly above a point measuring half its height.

From the foregoing, the determination of the pressure in a tunnel may be readily made.

If *L-O-K* is the roof of a tunnel (Fig. 10), in soil of which the depth is *A-O*, the pressure on *O-K* will be (but probably little more than) that due to the area, *O-M-K*.

The speaker assumes, however—and always with a factor of safety in view—that this pressure is due to the weight of soil below an area measured by $\frac{O\text{-}K \times O\text{-}K \tan S}{2}$, where $S = \phi + \frac{1}{2}(90° - \phi) = \frac{\phi}{2} + 45°$.

It is thus seen that by this determination full pressure of all the soil results when the depth of soil, *A-O*, is less than *S-O*, *S-O* being equal to $O\text{-}K \times \tan\left(\frac{\phi}{2} + 45°\right)$.

In this presentation of the principles as he finds them, the speaker hopes he has shown it to be possible to establish one basic formula for every condition of lateral earth pressure, one variable of which should be the height of trench, trench structure, or wall (or the width of tunnel), the other being based solely on the cohesive element of the soil in question, establishing therefor an angle of repose—and finding expression in terms of an angle of rupture—or an angle of internal friction, as determined.

E. G. HAINES,* M. AM. SOC. C. E.—The speaker has had the pleasure of reading the complete thesis presented by Mr. Feld, which is a very interesting document, as in it the author retraces the whole theory of lateral pressures from an early date to the present. The persistence with which the theory of the angle of repose has continued, whether right or wrong, as long as it agrees fairly well with observed facts, is of great interest. The angle of repose for soils was assumed by most of the early writers to be about 45°, and is now ascribed various angles corresponding to the characteristics of different soils. The author has re-stated the limits of such angles as 0° for water and 90° for rock. Although both are doubtless true within the limits in which the engineer usually is interested, the latter is not strictly true, as a vertical cut in solid rock, unsupported, will fail at some depth, as has been shown,† and the failure will not be vertical throughout, as has been substantiated by H. G. Moulton, M. Am. Soc. C. E.‡

The speaker has no particular fault to find with most of the theories for lateral pressure, or with tests made for ascertaining such pressures, except that when they are applied to soils in the natural state in large masses, it is found that the soils do not abide by the rules. Most of the tests have

* Asst. Div. Engr., Transit Comm., New York City.

† *Transactions*, Am. Soc. C. E., Vol. LX (1908), p. 27.

‡ *Transactions*, Am. Inst. of Min. and Metallurgical Engrs., February, 1920.

been made with reasonably dry sand, which is rarely found in practice. Sand is practically all silica; is fairly uniform in texture; has a low cohesive value; has a high frictional value; has normally a low water content; and, in short, is not only a granular but a granulated material.

The soils found in practice are composed of practically all the mineral elements derived from all classes of rocks, from the hard volcanic rocks and the limestones and sandstones to the softer clays and fine silts. Natural soils range from a dense, quasi-elastic mass, through the granulated stage, to an almost hydrostatic condition, and the degree of moisture or water content is variable. It is not reasonable to suppose that these soils will act in practice the same as sand under test conditions.

In arranging any device for measuring pressures, one must make some assumptions as to how the material will act, and when a machine is designed on those assumptions, dry sand readily accommodates itself to them. In making tests, using vertical shutter boards, it is invariably found that as sand is added behind the shutter boards and measuring instruments, a certain amount of pressure is obtained for a certain depth of sand. If a second similar depth is added, there is a similar increment, but it is also added to the first, etc. The resulting pressure diagram resembles a triangle standing on its base, at least at the time the test is conducted.

With soil in excavation, the process is reversed. As excavation is carried down step by step, there is a certain amount of pressure with each step, but it is also added to each one above. That is not theory; it is established fact, and may be observed on any deep timbered excavation in dry ground. The pressure diagram for excavation, therefore, also resembles a triangle, but its position is reversed, and it has its base at the top, at least at first. The speaker believes that Mr. Meem was the first to call attention to that fact.

The pressure diagram may not remain a true triangle in either the case of an embankment or an excavation; if there is a change in the water content or other conditions, the pressure diagrams change, but with an excavation, at no time, unless considerable water is present at the bottom, does the center of pressure ever transfer to a point much below the mid-height and, with embankments, there is evidence in some cases that the center of pressure is as high as the mid-height.

If material from behind a retaining wall is carried away by the drainage through weep-holes, or if the tide is carrying out or settling the material, the center of pressure becomes higher, and the upper material behind the wall will act as an arch. The reverse is true of an excavation. In the beginning there is no pressure, but the timbering as it is placed and keyed tight, transmits pressure into the bank, and the reaction of the bank which is a solid and quasi-elastic, converts the whole system into an arch, regardless of the direction of the stresses. Such pressure increases with depth of excavation. The contact between the timbering and bank is less perfect as the depth increases and, at some depth, the bank will fail, because of the shear from its own weight. When this occurs, there is a breaking up of the material, and as it approaches the granulated state there is also a corresponding re-

adjustment of the pressure, and it approaches, but never fully reaches, that of an embankment.

About three years ago, the speaker had occasion to investigate some cracks in the soil parallel to an excavation about 85 ft. deep. The crack was beneath a crushed stone or gravel sidewalk and was not apparent on the surface. Although the excavation had been made two years before, the crack was found by measuring outward a distance equal to one-half the depth of excavation. Measurements were made at a number of points for a distance of 1 000 ft., with the same success. That crack was so wide that one could stick his hand into it up to the wrist and had been there for more than two years, in material that one might class as sand. The surface of the crack was partly coated with the fine loam and stone dust brought down by rain water and melting snow. By means of a small shaft, the crack was traced downward for 26 ft. until it was lost in a vein of loose gravel. It is also safe to state that the crack could be traced at other points to-day, five years after the excavation was made. The point of interest is that, if, in this case, there was any arching action of the material from the side, why was not that crack closed? It should also be noted that the crack, instead of inclining at any angle—the so-called angle of repose—was practically vertical for the 26 ft. of depth; in fact, it inclined slightly outward, with increase of depth, although the material was one to which is usually ascribed an angle of repose of 1½:1.

There are many ideas and theories in regard to the pressures developed by soils. The speaker has contributed some of the theory himself, but as time goes on there is an inclination to discard one theory after another. Engineers have about reached a time when they will have to develop a new soils physics and a new theory of soil pressure, because most of the current ideas are wrong.

Soil is both the most common and the most complex construction material with which the engineer has to deal, and until it is subjected to analysis like other materials, and until its properties are much better known, there can be little hope for its successful use, and failures may be safely predicted.

One of the things that adds to the trouble is that engineers have borrowed from the layman names for broad general classes of soils and are attempting to use them to define certain definite materials. It does not follow that a so-called clay from the vicinity of the Great Lakes will possess the same characteristics as one from the Lower Mississippi Valley or from the arid Southwest, the upland shale district of West Virginia and Maryland, or from the glaciated metamorphosed rocks around New York City, for it certainly will not.

One of the most important new developments that has been brought out in the soils investigation recently, was presented to the Society in 1921 in a Progress Report* by the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations, etc., and elaborated somewhat

* *Proceedings*, Am. Soc. C. E., February, 1921, Papers and Discussions, p. 9.

by the presentation before the Society at its Annual Meeting in 1923 of samples of the colloidal element in clay. It is not known how far-reaching these discoveries will be, but the Committee hopes to study it with the help of the Society, and it is believed it will pay for the research, as there is little doubt that the presence or absence of that element constitutes a large measure of the stability of the soil, and its resistance to stress, either vertical or lateral.

GEORGE PAASWELL,* M. AM. SOC. C. E.—This paper is probably the best yet presented on the subject, from the standpoint of both the experimental data it contains and its thorough historical review. That it is consistent with well founded theory, makes one hope that it marks the end of the search for the proper method of determining the thrust on a retaining wall. A great many statistics have been accumulated, covering both experimental and theoretical research, and the author, in studying the accumulated data and assembling it, deserves more than the usual remarks of appreciation.

The speaker believes that there are two distinct problems connected with the determination of lateral earth pressure: (1) The retaining wall problem; and (2) the soil mechanics as evinced by the lateral pressure action. Experiment has been confined, to the present time, almost exclusively to Problem (1). What is the retaining wall problem? Does it vitally depend on the proper method of thrust determination, or are other factors of more importance? The experiments show conclusively that the Coulomb or a modified Rankine method gives the maximum pressure that can occur on a wall, taking a simple slope angle corresponding to that of a dry sand, namely, 30 degrees. With this value of the slope, and with the introduction of other approximations, the thrust expressions can be placed in very simple form. The speaker has shown† that the following two formulas give the thrust on a wall when the fill is supporting a surcharge which has been reduced to an equivalent earth fill the ratio of depth of which to the total depth of fill behind the wall is:

$$T = (1 + 2c)\ J\ (\tfrac{1}{2} w\, h^2)$$

$$T = (1 + 2c)\ K\ (\tfrac{1}{2} w\, h^2)$$

Table 36 gives the values of the thrust coefficients, K and J, for a number of slope values of the back of the wall. These coefficients represent the maximum thrust that can occur on a wall. The engineer, therefore, is justified in using these values to determine the thrust, just as in the design of other structures maximum loads are used, even though the actual loads are considerably less. There appears to be no justification for determining the exact values of the slope angle as these values are minute modifications that do not materially affect the formulas, and rarely occur in actual practice. Thus, the usual experimental material of dry sand is one most rarely found in practice, the usual fills consisting of loam, clayey materials, various undetermined mixtures, rock, and the like, for none of which a reasonably accurate slope value can be predicted. For this reason, the speaker feels that it is better to assume a

* Section Engr., Public Service Comm., First Dist., New York City.

† "Retaining Walls: Their Design and Construction."

thrust value based on a theory in reasonable accord with experiment, such as the values given previously, and then to proceed with the more important business of the design of the wall itself. That is the main problem; too often the designer deems the problem of the retaining wall solved when the thrust has been found. That is the easiest part of the problem. Examine the records of the failures of retaining walls, and note how few collapses are due to overturning moments. These remarks apply, of course, to the retaining wall built to carry an ordinary fill.

It seems reasonable to conclude that as far as the design of a retaining wall is affected, the exact determination of the earth pressure is an interesting, but not necessarily a vital problem, and the speaker feels that, in view of this conclusion, experimenters on lateral earth pressures should take more care to distinguish between the retaining wall problem and the so-called soil mechanics problem.

TABLE 36.—PRESSURE COEFFICIENTS.

β, degrees.	COULOMB COEFFICIENT, K.			Rankine coefficient, J.
	ϕ'			
	0°	15°	30°	
0	0.33	0.30	0.29	0.33
3	0.36	0.32	0.32	0.34
6	0.38	0.34	0.34	0.35
9	0.40	0.37	0.37	0.37
12	0.43	0.40	0.39	0.40
15	0.45	0.42	0.43	0.43
18	0.48	0.45	0.47	0.47
21	0.51	0.48	0.50	0.51
24	0.54	0.52	0.57	0.56

The author is correct in stating that a granular material is neither a fluid nor an elastic solid, but as it is obviously in a condition intermediate between these two, it should have some of the properties of its limiting conditions. Thus, it has been pointed out that elastic failure of solids may be explained on the theory of a constant angle of internal friction; that is, when materials start to fail under simple stress, failure surfaces will be found, that may be anticipated by assigning to the elastic solid an angle of internal friction of about 20 degrees. It is interesting to note that this introduction of an internal friction component into ordinary elastic theory seems to reconcile the granular and elastic solids, and, therefore, it seems reasonable to assume that some data on lateral earth pressure may be furnished by the theory of elasticity.

In conclusion, the speaker would state that, with no reflection on the value of the data presented, the paper adds nothing to the needed data on soil mechanics, and little more than is already known on retaining wall statistics. The rational bracing of a cut, and the proper design of a subsurface structure,

whether it is a large sewer, a subway, a tunnel, etc., awaits the development of a soil mechanics. The homogeneous granular materials of experiments are of little service in solving these problems.

D. B. STEINMAN,* M. AM. SOC. C. E.—The speaker desires to record his sincere appreciation of Mr. Feld's paper as an authentic contribution to the knowledge and understanding of lateral earth pressure phenomena. This paper helps to clarify a subject in which there has been much confusion and unscientific vagueness. A mass of literature has been published on the subject, with an abundance of mathematical derivations and ingenious graphic applications; but there has been a dearth of reliable experimental data on which to test the correctness or incorrectness of these basic assumptions. Most of the experimental work previously done in this field has been too crude or unscientific to yield any conclusive results. By examining the vast literature on soil mechanics, including the digesting and collating of more than 650 references, by designing and constructing an ingenious large-scale experimental apparatus for measuring earth pressures, by conducting an extensive and laborious series of tests scientifically planned and carefully executed, and by analyzing the results of those tests and reporting them in this excellent paper, Mr. Feld has rendered a real service to the Engineering Profession.

The author's experimental findings may be regarded as conclusive on a number of questions of fundamental importance, one of which is the direction of the resultant pressure on the back of a retaining wall. His experimental results appear to establish beyond further question the fact that this pressure is inclined above the normal to the back of the wall at an angle, ϕ', which ϕ' is the angle of friction between the wall and the retained material. The Coulomb theory, which assumes that the resultant pressure is normal to the back of the wall, and the Rankine theory, which assumes that the resultant pressure is parallel to the free surface of the fill, are thus overthrown. The author's results are also valuable in establishing the height at which the resultant pressure acts.

In his presentation of the distinctions between the angle of repose, the angle of internal friction, and the angle of internal resistance, Mr. Feld has contributed to a much needed clarification of some of the fundamental conceptions in soil mechanics; and in demonstrating that the angle of internal resistance is the proper angle to use in earth-pressure formulas, he has made a valuable new contribution to the application of earth-pressure theories. His findings on the effects of time, temperature, and surcharge on lateral earth pressure open many new questions for further study and experiment.

There are a few points in connection with the paper on which the author may be able to throw further light, one of which is the feature in the construction of the testing apparatus, whereby the side-wall friction was relieved. This feature is open to possible adverse criticism on the ground that it may have influenced the results to agree with a preconceived assumption. The detail consists of movable steel plates bearing on rollers against the side-walls and conforming in outline to the wedge of rupture. It may be con-

* Cons. Engr., New York City.

tended, with some justification, that this feature of the design assumes the theory of the wedge of rupture in advance, and that, consequently, any results obtained with this apparatus confirming that theory would be inconclusive. This construction may help to produce the condition assumed. The tests would have been more conclusive if some method had been projected to establish the validity of the wedge theory without resorting to a device which assumes that validity in advance.

In the conclusions from Mr. Feld's tests, there is one anomaly which deserves further study and investigation. In the tests on vertical walls, the measured pressures appear to confirm the Coulomb theory despite the fact that the same tests disprove the fundamental assumption on which that theory is based. The Coulomb theory assumes that the resultant pressure is normal to the wall, whereas the tests have proved this assumption to be wrong.

It appears paradoxical that the Coulomb theory which is based on the incorrect assumption of normal pressure, should fit the results of the tests better than the general wedge theory which embodies the correct assumption of inclined pressure. The author may have some explanation of this inconsistency, or further tests might help to eliminate it from the conclusions.

M. J. McPIKE,* ASSOC. M. AM. SOC. C. E.—In presenting his paper, the author showed that the pressures obtained on the scale-beams varied directly with the temperature. He stated that a movement of only $\frac{1}{10}\frac{1}{000}$ in. was the maximum movement of the gate for maximum pressure.

With such a small movement of the gate, it would seem that any slight variation in the length of the bell-cranks and connections of the apparatus itself would indicate pressures chargeable to expansion in the apparatus rather than to the soil, and this expansion would be a factor for consideration in the final results. It would be of interest to know whether any attempt was made to eliminate this factor.

It is to be hoped that the experiments will be carried to a further conclusion and that reliable data will be furnished for the proper design of walls with uniform back-fill as well as those with a combination of fills, and especially those with a water-bearing soil overlaid with a dry fill.

EDWARD GODFREY,† M. AM. SOC. C. E. (by letter).—Several things that stand out in any investigation of retaining-wall theories may be epitomized as follows:

1.—A vast amount of intricate theory and theoretical investigation is expended on this subject.

2.—All earth-pressure theories are based on a granular mass, generally without any assumption of cohesion.

3.—Grain and dry sand are about the only materials that are ever collected in sufficient quantities to require a container of large size, the nature of which materials fits the assumption of the theory.

4.—Grain and dry sand are kept in bins, but never as the fill behind a retaining wall.

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† Structural Engr. (Robert W. Hunt & Co.), Pittsburgh, Pa.

5.—Moist sand is seldom used as a fill, although it does not differ much from the material assumed by the theory.

6.—Loose garden earth has about the same properties as moist sand, but garden earth or soil is generally too valuable to be deposited behind a retaining wall, and if so placed, it would be compacted not loose.

7.—The sides of excavations in ordinary compact earth will stand alone. Unless much moisture is present, excavations are shored usually only as a precaution. The writer has seen pits for deep and large piers, largely undercut, excavated many feet into the soil without shoring.

8.—Coal which is perhaps the next material in order of importance to earth, that requires walls or bins for its confinement, can be made to stand several feet high in a vertical plane with a little care.

9.—The facts stated in Paragraphs 7 and 8 give legitimate grounds for the question, Why delve in fine theory in deriving a formula for a great force that, in fact and every-day practice, may be zero? Does not this kind of theorizing discredit the theory?

10.—A theory must be had for design, because it is known that any granular material will exert a great horizontal force on a wall, and that earth under certain conditions may also exert a great force.

11.—The real test of a retaining wall supporting earth fill occurs when the fill is saturated with water or is expanded by so-called frost action.

12.—No theory will codify the effect of these two conditions on a retaining wall, for the reason that no two walls are subjected to the same conditions in this respect.

13.—Experience with walls that have been built is the safest guide, in view of the impossibility of obtaining an exact theory. If this experience can be connected with a theory that is not too complex, that theory is a good guide for the extension of the rules of design.

14.—Tests on the lateral pressure of sand are valuable as a general guide in using a theory of earth pressures, but such tests should not be taken too seriously.

The natural corollary to these fourteen points is, that the only consistent theory for the design of retaining walls is one that approximates the results of tests and at the same time produces a wall corresponding in stability with successful walls. The simplest method of design is the equivalent fluid pressure method.

It is much simpler to determine the horizontal pressure against a retaining wall by assuming that the back-fill is a liquid the weight of which is some fraction of that of the soil. Such calculations are much easier to make and to check than those that necessitate the solving of a tedious formula, and when they are made, one may readily assure himself that they are correct.

The coefficient for the equivalent fluid pressure is all that is required in the solution of a problem in design where there is no surcharge. Tests are very valuable for cases of surcharge or superimposed load.

The equivalent fluid pressure method of design is not restricted to any theory, as it can be made to give results identical with the theories mentioned

by Mr. Feld, merely by using a suitable coefficient. Furthermore, this method of design does not mean that its user is under the hallucination that granular masses or earth act as liquids.

It is true that a fluid does not exert a vertical force parallel to the back face of a wall (the friction), and that strict adherence to the fluid pressure theory must neglect that force. It is true, also, that a vertical force is seldom considered in a design and, for safety, should be neglected.

On page 1453, Mr. Feld states that clay banks with overhanging tops and ditches with vertical sides are often seen, and that they are examples where cohesion is acting. Cohesion of earth makes the pressure of earth less than that of sand. Tests in which sand is used as a fill, therefore, are not proper criterions for judging the effect of earth fill against a retaining wall, unless those tests are interpreted with these facts in view.

In the next to the last column of Table 5, values are given that could be used as a coefficient for an equivalent fluid pressure. Neglecting the values for very small depths, it is seen that these coefficients are very regular, showing nearly constant results. The average is 0.264. This would be high for earth pressure, because if it is assumed that a stone wall of a width one-third its height is stable as a retaining wall, the coefficient for equivalent liquid pressure is about 0.17. Walls of these proportions are found to be stable. The conclusion, therefore, is that in the design of retaining walls to sustain earth fill the pressure of sand is greater than need be provided; and this is what is to be expected because of the cohesion of the earth.

It is significant to note that exactness in the lateral pressure theory, even for the regular sand used in these tests, is somewhat discredited by the results in Table 3. The horizontal pressures in Columns (13) and (14) should be alike, but there are large differences in these values. These differences amount to as much as 27 per cent.

On page 1494, under Conclusion 7, it is stated that it is better to use the wedge theory, disregarding the wall friction, and a complex formula is given for determining values of H and E. Why should E be found in this case, and why not at once reduce the problem to the equivalent fluid method and simplify all calculations?

One of the conclusions from Mr. Feld's tests given on page 1493 is that loads on the fill beyond the wedge of rupture have practically no effect on the lateral pressure. On page 1491, under "Conclusions", he states, "There is appreciable increase in pressure due to loads not on the wedge of rupture." This seems to be a direct contradiction of the statement on page 1493. This is an important matter, because the designer wants to know at what distance from a retaining wall a railroad track will exert lateral pressure against the wall.

On page 1492, Mr. Feld states, "Rolling or moving loads on a tamped fill have no effect on the pressure." Perhaps this is only the case where the load that rolls over the tamped sand is equal to or less than that which was exerted to tamp it. It would seem that a railroad load on a tamped earth fill would have a considerable effect on the pressure. Certainly, it would be poor engi-

neering to neglect the superimposed load of a railroad track in a retaining wall to support such a track. It would be helpful to the designer if it could be determined by test what effect should be computed for a load on the embankment, such as a railroad track? The writer would take the load per foot, say, 10 000 lb. for a locomotive, and divide it by 13, the width of the track, and use this 770 lb. as 7.7 ft. of surcharged earth, and would find the equivalent fluid pressure for this 7.7 ft. of earth. How would Mr. Feld solve the problem?

The author has worked out the coefficient to be used in the equivalent fluid method, although he does not so state it, and it would take some searching to find it. It is suggested that Mr. Feld, in his closure, discover whether one-half the vertical depth of the triangle of surcharge added to H will not give the horizontal pressure of his tests with about the same coefficient as that found without surcharge.

G. M. BRAUNE,* M. AM. SOC. C. E. (by letter).—As the writer was responsible for the conception and part-time direction of the experiments discussed by Mr. Feld, and in collaboration with Professor C. C. Myers, for the design of the apparatus used therein, it would probably add to the completeness of the paper if a fuller statement concerning the origin of the problem was given.

The validity of the theoretical earth pressure formulas had always been looked on with doubt by many engineers, as no experiments on a large scale had ever been undertaken prior to those made in Cincinnati. Several years ago, when the writer was engaged on the construction of some abutment wing walls, 37 ft. high, he recalls the discussion regarding the proper dimensions for such walls. One member of the engineering force, who was considered an expert in earth pressure formulas, after devoting several weeks of study to the proper design, submitted his computations to the resident engineer, who, like a number of others, did not have much confidence in earth pressure formulas. After much deliberation, he finally told the writer, who was associated with the contractor and was urgently requesting plans so that the work could proceed, that he would discard the theoretical computations and make the base four-tenths the height. Another point in the design of walls which was, and still is, under discussion, relates to the direction of the earth pressure; in other words, the question of whether there will be any frictional resistance between the filling material and the back of the wall. The late Professor Mohr, of Dresden, Saxony, contended that no such frictional resistance should be considered or would exist, whereas Dr. Mueller-Breslau held just as firmly that the resultant earth pressure was not normal to the back of the wall. Dr. Mueller-Breslau contended that as the wall was acted on by a lateral force, some slight movement would occur; and that however slight this movement might be, frictional resistance would come into action. Rankine's theory† assumes,

"* * * an incompressible, homogeneous, granular mass, without cohesion, the particles being held together by friction on each other; the mass being of indefinite extent, having a plane top surface, resting on a homogeneous

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† "Design of Walls, Bins, and Grain Elevators," by Milo S. Ketchum, M. Am. Soc. C. E.

foundation, and being subject to its own weight. These assumptions lead to the ellipse of stress and make the resultant pressure on a vertical wall parallel to the top surface."

When the wall is considered absolutely rigid, the Rankine theory holds true; and the resultant will be parallel to the top surface. As the wall is under stress, however, a slight movement will occur; therefore, on account of this movement, the conditions mentioned will be changed, and the resultant will no longer remain parallel to the top surface. In his paper entitled "Earth Pressures: A Practical Comparison of Theories and Experiments,"* L. D. Cornish, M. Am. Soc. C. E., questioned the assumption made in Rankine's theory that the direction of the resultant earth pressure is parallel to the top surface of the filling material. In discussing that paper, the writer alluded† to the experiments which he hoped to undertake at the University of Cincinnati, the results of which are now presented to the Society in Mr. Feld's paper. These, together with a number of similar, experiences convinced the writer that some extensive experiments should be conducted, and he decided that if an opportunity was presented, he would undertake such experiments on a large scale.

The writer became connected with the University of Cincinnati in 1912 and commenced the study of the proper design of apparatus. After reviewing a number of experiments that had been made, he finally adopted an apparatus similar to that used by Dr. Mueller-Breslau in his experiments at Charlottenburg in 1906. Dr. Mueller-Breslau used a free wall in front of the bin and determined the magnitude of the forces by measuring the deformation of supporting struts. His results—given in the discussion of the paper by Mr. Cornish referred to previously—show that the resultant earth pressure makes an angle of about 27° with the normal to the back of the wall and that its point of application is about 0.33 h above the base of the wall. His experiments included various slopes of surcharge, and his results showed that the direction of the earth pressure was not controlled by the slope. The Cincinnati experiments made on a much larger apparatus fully confirmed these results of Dr. Mueller-Breslau. The writer preferred a more direct method of measuring the forces than those adopted by Dr. Mueller-Breslau, and enlisted the services of Professor Myers in the design of the Cincinnati machine. Together, they designed an apparatus substituting platform scales for measuring struts. Complete details of this apparatus were worked out by B. H. Wolfkoetter, a Fellow of the Civil Engineering Department, under the writer's direction, and were completed in 1916. Further work on this problem was delayed on account of the World War. In 1919, Mr. Feld came to the writer as Baldwin Fellow in the Civil Engineering Department, and the suggestion was made to him that he complete this problem and submit it as a Doctor's thesis. Under the writer's direction, Mr. Feld made a new set of detailed drawings of the apparatus which embodied, however, few fundamental changes in the original design. Owing principally to the high price of steel,

* *Transactions*, Am. Soc. C. E., Vol. LXXX (1917), p. 191.

† *Loc. cit.*, p. 218.

and also with the idea of obtaining a more rigid structure, the writer made the bin of concrete, and for the same reason, made the gate of wood, instead of steel. In order to eliminate the frictional resistance between the filling material and the sides of the bin, the writer conceived the idea of permitting the sides of the bin to move with the filling material. This idea was an error and in making the detailed drawings of the apparatus as finally built, only that part of the side-walls was made movable that was in direct contact with the breaking prism.

The writer received valuable aid and suggestions from Professor Cain, when he first undertook the design of the apparatus, and at his suggestion the front wall was made 6 ft. high, as this height was considered the minimum to adopt in order to obtain practical data. In discussing this problem with Dean Ketchum, the writer's attention was called to an important point that probably might have been overlooked in making these experiments; that is, whether the scales would record active or passive pressures. In order to eliminate any uncertainty, Dean Ketchum suggested that if the riders on the scale-beams were moved out and the scales over-balanced, the pressures that would be recorded when the scale-beams were permitted to float, would be the active, and not the passive, pressures.

The writer wishes to take this opportunity to express his appreciation of the perseverance and conscientious work that was done by Mr. Feld in conducting these experiments, and he believes that the publication of this paper in the *Transactions* of the Society will be extremely valuable to the Engineering Profession.

THOMAS H. WIGGIN,* M. AM. SOC. C. E. (by letter).—This paper has much merit, not only for its store of well digested experimental data, but also for its painstaking correlation of the new data with the results of previous researches on the same subject.

The writer would suggest the following explanations, many of which are not new:

1.—The direction and point of application of resultant earth pressure depend on the kind of support, that is, its degree of rigidity; also on the state of stress in the earth at the time equilibrium is reached between the pressure of the earth and the resistance of the wall. The author's experimental walls, of course, received only the "active" pressure of newly placed sand. Trench bracing may be so strong and so wedged as to develop part of Rankine's "passive" pressure, or may be so weak and slack as to compel the earth to support itself in part by cohesion, such cohesion being greater than that assumed possible in earth by any of the theories under consideration.

2.—With rigid support the pressure on a vertical surface from a level field of earth will be exactly horizontal. This seems to be evident from a consideration of the symmetry that must exist in undisturbed earth supported by itself. Manifestly, conditions are the same on each side of any vertical

* Cons. Engr., New York City.

plane, so that there is no reason for the earth pressure to slope more in one direction than in another.

3.—If, in a level field, one could substitute a rigid wall for the earth on one side of a vertical plane, without disturbing in the least the state of stress in the earth, the pressure on that wall would be exactly horizontal.

4.—In earth-pressure experiments like those of the author, the wall is placed first and the earth is then shoveled in behind it. The wall is practically incompressible vertically, and, through the agency of the friction of the earth on the wall, retains the adjacent earth in suspended condition. This tends to produce Mr. Meem's arching effect, with the wall for one abutment; there is, of course, a vertical component from the arch thrust, and this component is registered on the scales provided by the author in his experiments. In other words, the resultant in this case is inclined, as the author proves.

5.—The fill against a retaining wall is generally placed by the same method as that used by the author; hence, the experiments give correctly the nature of the thrust against the ordinary retaining wall. A retaining wall must rock forward, however, to sustain its load (although in rare cases it may settle more at the heel than at the toe); hence, the author's wall, supported rigidly at top and bottom, cannot entirely represent the ordinary wall. The Goldbeck cell for measuring pressures in earth offers a better means than was formerly available for observing pressures and changes of pressure with lapse of time in earth back of actual structures. It would be extremely valuable to have long-time observations on actual structures, and a parallel set of the author's tests, using the same earth.

6.—In a sheeted trench, the pressure will depend on the method of sheeting and the care taken to prevent earth movement. A trench in moderately cohesive earth may be braced by longitudinal planks and cross-struts. In this case, the sheeting cannot carry a vertical component and the earth has to continue to do so, as it did before the trench was dug. In other words, pressure on the support is horizontal. With vertical sheeting, on the other hand, particularly when not wedged tightly, vertical earth loads may be thrown on the sheeting; that is, the resultant earth pressure may be inclined.

7.—Mr. Meem has called attention to the fact that for certain kinds of earth, trench support may be designed successfully on the assumption that most of the earth pressure comes near the top of the trench. If the earth has some cohesion, as is usual, the section of earth which tends to slip down, when the support of the earth which was in the trench is removed, may be held up by various combinations of braces. Manifestly, a support anywhere in the depth of the trench would hold up a wedge of cohesive material. The brace, *A*, in Fig. 11, might do all the work, if cohesion at and below *cd* were sufficient; or the brace, *B*, might do all the work or only a small part of it. As earth is of many degrees of cohesion and plasticity, trench bracing must be adapted to the material encountered, and will never be solved by any universal rule except one that would be extravagant in most cases.

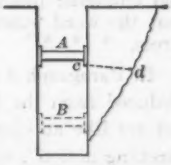


FIG. 11.

8.—In connection with the slides of the Panama Canal, the formation of vertical cracks has been explained mathematically by Professor Slichter, if the writer remembers correctly. A high bank of cohesive earth or weak rock, Fig. 12, fails by shear along surfaces which are vertical at the top and curve to a nearly horizontal direction near the bottom. All deep trenches show this tendency, which, of course, is more noticeable when the support is inadequate or the braces are not tight enough. It is not a wedge, but a semi-segment, that is to be supported in these and perhaps in most cases.

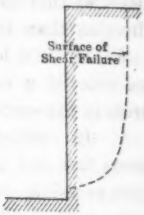


FIG. 12.

9.—After the pressure of various kinds of earth has been measured satisfactorily, it remains—in colder climates—to determine the effect of freezing at the tops of retaining walls. The retaining wall will continue to be a structure requiring local experience as well as sound judgment, but that does not diminish the value of the author's completed and prospective work.

F. N. MENEFEE,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper is a real contribution to experimental information on the subject of lateral earth pressures, and shows a commendable amount of work, both in carrying on the experiments and in correlating the data.

The time has almost come when the Society should consider the more up-to-date data, standardize definitions and symbols, and codify the results of research and discussion in such a way that they will be of greatest benefit to the profession.

Some of the terms used in this paper are well chosen, but are not yet accepted definitely by engineers. The writer refers to the author's "coefficient of internal resistance" and "coefficient of internal friction". As soon as engineers can decide on definitions which are more or less self-explanatory, and terms which admit of only one interpretation, earthwork problems will be solved quickly with the aid of the data on the subject available to-day.

In a discussion† of the Progress Report of the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundations, etc., for 1920, the writer suggested that the term "elastic limit" be not used in discussing the compressibility of earth, for the following reasons:

"* * * that for clearness and definiteness this term should not be used in discussions of materials which have little or no tensile strength. An analysis of the term 'elastic limit', or 'limit of elasticity', leads one to think of an inherent and enduring quality, at least unchangeable by the weather, say. It seems that the word 'elastic' implies complete restoration of shape with removal of stress, * * *."

In Paragraph 2 of his "General Conclusions on the Pressure of Sand Fills, Deduced from the Cincinnati Experiments", Mr. Feld states: "The fill does not act like an elastic solid", and he is correct. However, he uses the term, "arching action", which the writer thinks will die hard, yet he believes it has been proved that the phenomenon which suggests itself when the term is used,

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† Pamphlet, Am. Soc. C. E., January, 1921, p. 12.

does not happen. That is, superimposed loads do not produce stresses in the direction indicated by Fig. 13, but rather in that indicated by Fig. 14. The reader is referred to the writer's discussion on the Progress Report of the Special Committee on the Bearing Value of Soils for Foundations, etc., previously mentioned.

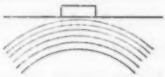


FIG. 13.

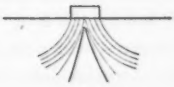


FIG. 14.

Mr. Feld's outstanding conclusion: "The resultant pressure is inclined to the wall, deviating from the normal by an angle equal to the angle of friction between the fill and the wall", has been used by the writer for several years, and is the most rational and direct method of determining the maximum slope of earth pressure to the normal. For a theoretical discussion of the conclusion advanced by Mr. Feld, the reader is referred to the discussion* by the writer of a paper entitled, "Earth Pressures: A Practical Comparison of Theories and Experiments" by L. D. Cornish, M. Am. Soc. C. E.

CHARLES TERZAGHI,† ESQ. (by letter).—From the very fact of their being essential, the essential features of any phenomenon are exhibited, even in case the phenomenon is produced on a miniature scale. Therefore, the apparent contradictions between the results furnished by the Cincinnati tests and the conclusions previously drawn by the writer from his own experiments touch the most vital points of the problem.

Effect of the Movement of the Wall.—A brief summary of the writer's conception of the theory of earth pressure against retaining walls has been presented by Mr. Feld.‡ The coefficients determined by the writer were mentioned by Mr. Feld merely for the sake of completeness, and not because he attributes to them any quantitative value. Nevertheless, the writer claims that they are sufficiently accurate to serve at least as a starting point for the discussion of the results of even such elaborate tests as those made in Cincinnati. They cannot be considered as conclusive, but they give a fairly clear idea of the phenomenon as a whole, and no very important difference is to be expected between them and the values furnished by large-size tests.

According to the writer's conception, the problem of computing the earth pressure against a retaining wall is statically indeterminate, the earth pressure being a function of the distance over which the wall yields. The earth

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 218.

† Cons. Engr.; Assoc. Prof. of Civ. Eng., American Robert Coll., Constantinople, Turkey.

‡ On pp. 1503–1505. The description of the writer's method of measuring the earth pressure at rest is not quite correct, inasmuch as the measuring tape did not rest against another tape of the same kind, but slipped between two strips of special paper. Thus, an exceedingly small pressure was sufficient to cause the tape to "bind". A full description of the apparatus is contained in the writer's article, "Old Earth-Pressure Theories and New Test Results", *Engineering News-Record*, September 30, 1920. The statement, "the earth pressure against a perfectly rigid wall is fairly independent of the density of the back-filling", obviously concerned the ratio, $\frac{H}{\frac{1}{2} \gamma h^2} = 0.42$, otherwise, the formula. $H = 0.42 \frac{\gamma h^2}{2}$, could not possibly contain the specific gravity, γ.

pressure acting against a perfectly rigid wall should be of the order, $0.42 \frac{y\ h^2}{2}$.

The coefficient, 0.42, was determined for crushed and sifted quartz, and was found to be independent of whether the sand was loose or compacted. It may be somewhat different for other kinds of sand. If the wall yields, the pressure should decrease rapidly and be a continuous function of the distance covered by the wall before the first "slip" occurs.* At the same time, the distribution of the pressures over a vertical strip of the wall should deviate more and more from the straight-line law, and the point of application of the resultant pressure should move upward from the lower third of the height. This process was called by the writer the "first phase" of the earth pressure, and involves a decrease in lateral pressure of about 48% to 31% of the original value, according to the density† of the structure of the sand.

During an early stage of the first phase, the lateral expansion of the back-filling was found to be an elastic one, and the distance of yield required for approaching the second phase, based on a knowledge of the modulus of elasticity of the sand, can be roughly computed. Table 37 shows the moduli of elasticity, E_s, determined by the writer for the compression of different granular materials at a constant cross-section, p denoting the vertical pressure, in kilogrammes per square centimeter, acting on a horizontal cross-section within the mass, and p_i an "initial" pressure which, for back-filling with a height of $h < 10$ m., is about 0.14 kg. per sq. cm.‡ The lateral expansion of the back-filling involves in addition a settlement in a vertical direction, hence a deformation in two directions, and the modulus of elasticity increases, according to Table 37, directly with the depth below the surface. Considering these two facts the horizontal movement of the wall, s_1, was computed,§ which is required for reducing the horizontal component of the lateral earth pressure exerted by a fill with a height of 72 in. (183 cm.) with 10% of its original value, $0.42 \frac{y\ h^2}{2}$.

The values of s_1 thus obtained are presented in Table 37. Of the materials mentioned in that table, the pure quartz sand seems to be most like that used in the Cincinnati tests. For this reason let the value of $s_1 = 0.00483$ cm. be

* The first experimental record of the fact that the intensity of the earth pressure decreases with the distance, over which the wall yields, is contained in a paper published by G. H. Darwin (*Minutes of Proceedings*, Inst. C. E., Vol. LXXI, 1882–83, Pt. 1). A. Donath has observed, that the intensity of the earth pressure decreased from 18 to 20%, if the wall (15 in. high) was allowed to turn through an angle of 1′ around its lower edge (*Zeitschrift für Bauwesen*, 1891, p. 491). K. Skibinsky concluded the pressure-relieving effect of wall movements from his theory of the equilibrium of back-fillings, consisting of a mass of rough spheres ("Das Gleichgewicht des rolligen Materials", *Oesterreichische Wochenschrift für den öffentlichen Baudienst*, 1916).

† The intensity of the earth pressure which acts at the first slip, depends on the density of the sand, while the earth pressure at rest seems to be independent thereof. Hence, the difference between these two values depends on the density of the sand.

‡ For a very loose quartz sand, p_i was found to amount to about 0.07 kg. per sq. cm., and for a compacted sand it increases to about 0.21 kg. per sq. cm. The value, 0.14, therefore, represents the average.

§ For a detailed description of the elastic properties of granular masses and for the method of computing the modulus of elasticity, E, for the compression of a material which is free to expand laterally (for instance, the back-filling of a yielding retaining wall), see the writer's paper, "Die Beziehungen zwischen Elastizität und Innendruck", *Sitzungsberichte* der Wiener Akademie der Wissenschaften, 1923 (In print). The values of E_s (Table 36), are presented merely for the purpose of giving an idea of the limits, within which the modulus of elasticity of dry, granular materials may possibly vary.

retained for use in the discussion. From the diagrams published by the writer, it may be learned that the first slip involves a total horizontal outward movement of the wall of about $0.0010\,h$,* which in the case under consideration would be $0.0010 \times 183 = 0.183$ cm. Using these data as a basis, the relation between the horizontal movement of the Cincinnati wall and the ratio, $\frac{H}{\frac{y\,h^2}{2}}$, has been plotted approximately in Fig. 15 (a). Starting with an intensity of $0.42\,\frac{y\,h^2}{2}$, the decrease of the earth pressure, during an early state of the first phase, is determined by the tangent, $r\,t$. Toward the end of the first phase, the curve turns abruptly downward and terminates in the point, s_1 (first slip).

TABLE 37.

No.	Kind of material.	E_s, in kilogrammes per square centimeter.	s_1, in centimeters.
1	River sand with rough grains....	176.0 $(p + p_4)$	0.00276
2	Pure quartz sand.....	100.0 $(p + p_4)$	0.00483
3	Sand with silt and clay.....	94.3 $(p + p_4)$	0.00511
4	Dry, pulverized clay.....	80.7 $(p + p_4)$	0.00597

E_s = modulus of elasticity of the material, lateral expansion being prevented. It refers to the thickness which the layer of sand would have if the volume of voids is zero.

p = pressure, in kilogrammes per square centimeter.

s_1 = yield of a rigid retaining wall, 6 ft. (183 cm.) high, required for producing a decrease in pressure of 10% at the outset of the first phase.

The results of Mr. Feld's experiments for the purpose of detecting the influence of the movements of the wall are given in Table 1.† A horizontal outward movement of ($\frac{1}{400}$ to $\frac{1}{1600}$ in.) = 0.00634 cm. caused a decrease of the horizontal component of less than $\frac{1}{3}$ (3.78% + 3.02% + 3.41%) = 3.4%, and the horizontal component amounted to $0.25\,\frac{y\,h^2}{2}$. Hence, at the same rate of decrease, the reduction of the earth pressure by 10% of $0.42\,\frac{y\,h^2}{2}$ would have

* According to the writer's conception, a definite decrease of the lateral pressure has its equivalent in a definite change of the structure of the sand, that is, in a change in the arrangement of the grains. Hence, for geometrical reasons, there should exist for any sand a simple proportionality between the height, h, of the back-filling and the distance, s, of the yield required for reducing the lateral pressure with a definite percentage. One arrives at the same conclusion if the investigation of the relation between h and s is based on the elastic properties of the sand.

† The writer understands the data in Table 1, to be as follows: The difference between the first set of readings (scale-beam in a restricted position) and the second set (scale-beam at the highest point of the trig loop) corresponds to an outward movement of the wall over a distance of $\frac{1}{400}$ in. horizontally. By pushing the wall back to its original position, the pressure resumes approximately its original value. As it is possible (although not mentioned in the paper), that that part of the difference between the first and the second scale-reading was due to the construction of the scale, the value of 3.4% should be considered as a limiting value (upper limit of the pressure-relieving effect of the wall movement). According to the writer's conception (Fig. 15(a)), the decrease of the lateral pressure, produced by a wall movement over a distance, s, approaches the value, 0, before the first slip occurs. The outward movement of the upper edge of Donath's retaining wall (15 in. high, mentioned previously) produced a decrease of the lateral pressure of 18 to 20 per cent. In a more advanced state of the first phase, the pressure-reducing effect of such a movement should have been equal to zero.

required a yield of at least

$$0.00634 \times \frac{10.0}{3.4} \times \frac{0.42}{0.25} = 0.0313 \text{ cm.},$$

which is 6.50 times more than the writer has estimated. This is a considerable discrepancy. Far more important, however, is the fact that the earth pressure measured by Mr. Feld amounted to not more than $0.25 \frac{y\,h^2}{2}$, although the total horizontal movement of the wall was only $\frac{1}{1000}$ in. = 0.000159 cm., which is negligible, even if compared with the values of s_1 presented in Table 37. According to the writer, the horizontal component should have been about $0.42 \frac{y\,h^2}{2}$. This obvious contradiction makes him suspect that the elastic deformations of the wall itself may have been so important that its movement as a whole was a negligible item.

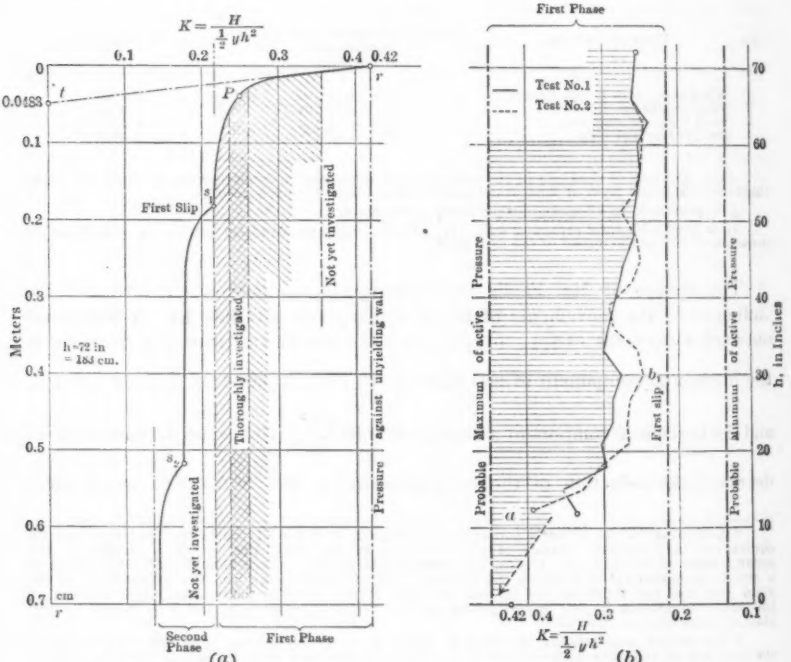


FIG. 15.

The two lower horizontal contacts were 1.44 m. apart. They supported a horizontal wooden stringer of 10.2 by 15.3 cm., which, in turn, served as a support for the lower ends of the $\frac{3}{4}$-in. sheathing and the four 3 by 6-in. vertical beams. At the outset of the experiment, the horizontal pressure acting against

that timber was equal to zero, and, when the bin was filled, it increased to about $\frac{2}{3} \times 1\,040 = 693$ kg., or $\frac{693}{153} = 4.53$ kg. per cm. of the length of the beam.

Let,

$I = \frac{10.2 \times 15.3^3}{12} = 3\,050$ cm.^{4}, be the moment of inertia of the beam;

$E = 110\,000$ kg. per sq. cm., the modulus of elasticity of the timber;

$l = 144$ cm., the span of the beam; and,

$q = 4.53$ kg. per cm., the load per centimeter.

The maximum deflection of the beam, therefore, was, approximately,

$$f = \frac{5\,q\,l^4}{384\,E\,I} = \frac{5 \times 4.82 \times 144^4}{384 \times 110\,000 \times 3\,050} = 0.07 \text{ cm.} = \frac{1}{36.3} \text{ in.}$$

That is a large quantity when compared with the distances, s_1 (Table 37), required to produce a decrease of 10% in pressure. With Mr. Feld's figures (decrease of the lateral pressure equal to, or less than, 3.4%), the writer obtains for the upper limit of the pressure-relieving effect of a deflection of the magnitude mentioned previously $\left(\frac{1}{36.3} \text{ in.}\right)$, the value, $3.4 \times \frac{0.07}{0.00634} = 37.5$ per cent.

Thus, the crucial point for the whole further discussion depends on the comparative magnitude of the elastic deformation of the wall and its important influence on both the intensity and the distribution of the pressures.

Starting again from the writer's conception of the earth-pressure phenomenon: The effect of the deflection of the beam on the results of Mr. Feld's tests can be predicted approximately.

Let,

h = the variable height of the fill;

l = the distance between supports;

$H = k\,\frac{y\,h^2}{2}$, the horizontal component of the earth pressure, both k and h being variable; and,

$s = \frac{5\,l^4}{384\,E\,I} \times \frac{2}{3} \times k\,\frac{y\,h^2}{2} = c\,\frac{y\,h^2}{3}$, the deflection of the beam at the bottom of the gate.

According to the data published by the writer, the ratio, k, is a function of the ratio, $s : h$, between the amount of yield of the wall and the height of the fill. The character of the function which connects k and s is represented graphically in Fig. 15 (a) and can be expressed by the symbolic term:

$$k = 0.42 - \alpha\,\frac{s}{h} + f\left(\frac{s}{h}\right) = 0.42 - \alpha\,\frac{c\,y\,h}{3} + f\left(\frac{c\,y\,h}{3}\right) \ldots\ldots (1)$$

α is a constant, and $f\left(\frac{s}{h}\right)$ is an arbitrary function which suits the follow-

ing conditions: In an early state of the first phase, both the function and its first derivative are negligible, and in an advanced state, both the function and its first derivative increase. The item, $f\left(\frac{s}{h}\right)$, simply expresses the fact that the proportionate effect of the movement of the wall on the intensity of the earth pressure decreases rapidly in an advanced state of the first phase. If a constant average value for c is assumed, the relation that exists between k and h is obtained. The expression,

$$\frac{d\,k}{d\,h} = \frac{\alpha\, c\, y}{3} + \frac{d\, f\,\left(\frac{c\, y\, h}{3}\right)}{d\, h} \qquad (2)$$

Thus, in an early state of the experiment, while the height of the fill in the bin is still small, the second term of the right-hand member is negligible, according to the assumption, $\frac{d\,k}{d\,h}$ is negative and k decreases. As the height of the fill is increased, the phenomenon enters an advanced stage of the first phase, and the second item comes into prominence. For

$$\frac{\alpha\, c\, y}{3} = \frac{df\,\left(\frac{c\, y\, h}{3}\right)}{dh}$$

becomes stationary and then $\frac{d\,k}{d\,h}$ assumes even positive values, that is, k increases. These facts come clearly into evidence from the figures communicated by Mr. Feld in Table 5. His values of $K = \frac{H}{\frac{y\, h^2}{2}}$ are graphically represented in Fig. 15 (*b*). In fact, the ratio, $\frac{d\,k}{d\,h}$, changes the sign at the point b_1; the close agreement between theory and observation bears splendid testimony to the care and conscientiousness with which the tests were made. Nevertheless, a more detailed study of the curve, including its upper part, is impossible because, from a static point of view, the case is hopelessly complicated. After the depth of the fill exceeds half the depth of the bin, the upper horizontal beam comes under important pressure as well, and deflects, causing a renewed decrease of the value, k. Furthermore, the deflections are not distributed uniformly over the whole gate; they are practically equal to zero at the contacts and are a maximum at the centers. Thus, one part of the back-fill is almost at the beginning of the first phase, while other parts on the same horizontal line, are approaching the second phase.

The lack of uniformity in the distribution of the pressures along the horizontal lines of the gate represent such an important distinguishing feature between the pressures acting against the Cincinnati gate and those acting against an ordinary retaining wall, that no conclusion whatsoever can be drawn from the results, unless the deformations of the gate are determined and the

actual distribution of the pressures over the gate is investigated with pressure cells or similar devices placed near or built into the gate.

A general idea of the distribution of the pressures horizontally can be obtained in the following manner: Compute the deformations of the wall under the influence of the load, in case the load is uniformly distributed over horizontal strips of the gate, and then measure the actual deflections. From the difference between the two sets of data the distribution of pressures can be calculated approximately by a graphical method. The writer practiced this proceeding successfully when he investigated the distribution of the pressures over the elastic side-walls of the model for a reinforced concrete storage bin. It would be of interest to have both the calculated and the measured deflection published. It is likely that the results will furnish an instructive example of the relations which exist between pressure and yield. These relations have been shown clearly in the results obtained from miniature tests—simply because they are an essential fact.

Effect of Temperature.—The effect of changes in temperature on the intensity of the earth pressure is a self-evident consequence of the relation just discussed. A drop in temperature has the same effect as the yielding of the wall of an amount equal to the contraction of the "wedge"; and the intensity of the effect, therefore, depends on the "phase" in which the drop in temperature occurs. During the initial stage of the first phase ($k = 0.42$ until $k \doteq 0.3$), a movement of the wall of 0.00483 cm. causes a decrease of 10% in the lateral pressure. The average coefficient of linear expansion of rock grains is 0.000005. In half the height of the fill the width of the wedge is equal to about 45.5 cm., hence a drop in temperature of 1° cent. causes an average contraction of $0.000005 \times 45.5 = 0.000227$ cm., and a decrease in pressure of

$$10.0 \times \frac{0.000227}{0.00483} = 0.470 \text{ per cent.}$$

Mr. Feld's observations were made in an advanced stage of the first phase (Point P, Fig. 15 (a)), when the curve no longer coincides with the tangent, rt. At this stage, and according to the figures in Table 1, a decrease of 10% in the lateral pressure acting at $k = 0.25$ requires a movement of at least 0.313 $\frac{0.25}{0.42} = 0.0187$ cm. of the wall. Hence, a decrease in temperature of 1° cent. causes a decrease in pressure of less than $10.0 \frac{0.000227}{0.0187} = 0.121$ per cent. Mr. Feld observed 0.15 per cent. As the observed effect of the temperature is even greater than the upper limit of what can be accounted for by the expansion of the wedge, the balance may be due to the effect of the temperature on the gate and on the pressure-transmitting device. At the end of the first phase and during the second one, the effect of temperature should become negligible, because the effect of lateral expansion on the intensity of the earth pressure becomes very small, as compared with the effect during the first phase.

Internal Resistance and Internal Friction.—Considering the fact that even the large-scale Cincinnati observations confirm the existence of intimate relations between the intensity of the earth pressure and the distance of yield or

deflection of the wall,* the question arises: Wherein consists the internal friction? For the back-filling of a perfectly rigid wall, the angles of internal friction can be computed from the ratio:

$$\frac{H}{\frac{y\,h^2}{2}} = t\,g^2\,\frac{90 - \phi_0}{2} = 0.42,$$

which gives $\phi_0 = 24$ degrees. As already mentioned, ϕ_0 for other kinds of sand might be somewhat different than for crushed quartz. For a yellow plastic loam, the writer found that $\phi_0 = 10° 0'$, and for a blue plastic clay, $\phi_0 = 8° 20'$. The general trend of the curve in Fig. 15 (*b*) suggests that the value of ϕ_0 for the sand used by Mr. Feld was rather smaller than 24 degrees. If the wall yields, the value of ϕ increases continuously until it assumes a maximum at the end of the second phase. The value, ϕ_2, was found to depend largely on the original density of the structure of the sand. The writer obtained from his experiments:

Sand	$\phi_2 = 47° 40'$ to $64° 40'$,	$\tan \phi_2 = 1.098$ to 2.112
Yellow plastic loam.....	$\phi_2 = 22° 20'$,	$\tan \phi_2 = 0.411$
Blue plastic clay.....	$\phi_2 = 13° 40'$,	$\tan \phi_2 = 0.243$

These figures give at least an idea of what can be expected to be the upper limit of the internal resistance, in spite of the fact that they are not conclusive.

At an angle, ϕ_1 ($\phi_2 > \phi_1 > \phi_0$), the first "slip" (s_1 in Fig. 15 (*a*)) occurs. It was defined by the writer to be "a radical but temporary expedient to remedy a local disorder in the structure of the sand."† It is a non-essential event, because there are many other earth-pressure phenomena which do not involve any slip or at least no slip along the plane for which the angle between the resultant stress and the normal to the interface is a maximum. The value of ϕ_1 may range between the limits of 40° and 52°, according to the original density of the sand.

Hence, it is necessary to distinguish between two essentially different kinds of internal resistances:

(*a*) The internal friction, which is a function of the amount of yield of the wall, and determines the intensity of the earth pressure for any stage except for the slip. While the wall is yielding, it increases from the initial value, ϕ_0, to the maximum, ϕ_2, whereby ϕ_2 is at least twice as large as ϕ_0. This type of friction is by no means the resistance to "shearing" along a definite plane, but is merely the resistance to a mutual displacement of the sand grains throughout the space.

(*b*) The frictional resistance to separation along a plane. This resistance cannot possibly develop unless there is a tendency to separate along a definite plane.

The stage of lateral expansion at which Mr. Feld made his observations was found to correspond to a point, *P*, situated within the first phase of the

* See the text referring to Fig. 15 (*b*) (Equations (1) and (2)).

† *Engineering News-Record*, September 30, 1920, p. 637.

earth pressure, and on the downward slope of the curve, Fig. 15 (*a*). In this stage, the first slip has not yet occurred. In fact, Mr. Feld states, "there is no sharply defined wedge of rupture, nor can any surface of rupture be detected on the surface of the fill" (as a matter of course, because the point, s_1, was not yet attained). Nevertheless, he defines the internal friction as "a resistance against shearing along a plane" and both his "internal resistance" and his "internal friction" are of Type (*b*). If, furthermore, the "internal resistance", ϕ_i, mentioned by Mr. Feld, is in fact a resistance to separation along a plane, and, in addition, is smaller than the tangent of the angle of repose, tan ϕ_r, the natural slope could not possibly be stable. Along each plane that intersects the natural slope and forms with the horizontal an angle, ϕ, whereby $\phi_r > \phi > \phi_i$, there exists a shearing force which, according to the laws of statics, should lead to a slip. Mr. Feld, however, has measured both ϕ_i and ϕ_r; he found $\phi_i < \phi_r$, and, in spite of that, the natural slope remained stable. (For instance, see Table 14: Angle of natural slope = 32° 45′, internal resistance = 29° 40′, and internal friction = 27° 30′.)

The cause thereof is in the manner in which the coefficient of internal resistance was measured. The resistance to pulling a cubical bottomless box, filled with sand, over the level surface of a mass of sand is an exceedingly complicated phenomenon involving a passive earth-pressure effect within the box. It has nothing to do with a simple frictional resistance. A rectangular box would furnish altogether different values, but of the same quality.

Mr. Feld's "internal resistance" differs from "internal friction", because it includes cohesion. He states that the cohesion resistance "is equal to a constant times the area of the surface." Hence, it is not quite clear how it can appear in his calculations as an angle. Thus, for instance, in Test No. 1: The angle of internal resistance = 37°, and that of internal friction = 34 degrees. If p denotes the pressure per unit of surface, the difference between the internal resistance and the internal friction would be, p (tan 37° — tan 34°) = 0.079 p, that is, proportional to the pressure, p.

Furthermore, one feels compelled to hesitate, when reading:

"It is found that two coefficients may be determined, one corresponding to the force required to start the mass in motion, the other corresponding to the force required to keep the mass moving uniformly," and "it should be noted that, for the dry sand, there is no difference between the angles of friction and resistance."

The internal friction depends on two qualitatively different factors: On the coefficient of the friction which acts along the surfaces of contact of the grains, and on the manner in which the grains are arranged. The effect of the arrangement of the grains on the internal friction was anticipated by Clark Maxwell, who called it "the influence of the historical element". It can be studied theoretically by analyzing the internal resistance of a mass of rough spheres; it was demonstrated experimentally for the first time by Darwin in 1877, and it is, after all, a self-evident fact, because, if the arrangement of the grains had no influence on the internal friction, it would be impossible to build a stone wall without using mortar. Hence, the internal friction in a thoroughly compacted mass of sand must be considerably different from that

which, under equal circumstances, acts within a very loose mass of sand. The phrase, "equal circumstances", means an equal degree of lateral expansion. The classical experiment by O. Reynolds, in 1885, with the rubber bag filled with wet sand, showed that no mass of sand can change its shape, that is, undergo any radical internal movement, unless its volume of voids previously assumes a maximum. However, according to Mr. Feld, the resistance in starting the dry sand in motion and the resistance in keeping it in motion are identical—this means that he identifies the internal resistance of the back-filling with the resistance in a very loose sand, no matter whether the back-filling is loose or compact.

If the sand is wet, the only differenee between the resistance against starting the motion and the one required in maintaining it, should be, according to Mr. Feld, the presence of cohesion in the former case and its absence in the latter. He states: "In dry sand there is no cohesive force; the attraction is then between the grains of earth and the water"; but, as the wet sand continues to remain wet, after the motion has started, the writer cannot understand why the attraction should be suddenly absent. Furthermore, the writer is astonished to learn that the physical cause of the cohesion of a wet sand should be a "doubtful point". It is a well-established fact that "a surface attraction between unlike particles or an adhesion" always exists, but, at the same time, it is a completely negligible factor, unless the particles are of colloidal size; and the attraction between wetted surfaces of a macroscopic size is merely due to the surface tension of the liquid.

Let,

F = the area of the wetted surface;
T = the surface tension of the water; and
d = the thickness of the film.

The attraction between the two surfaces is equal to $P = \frac{2\,F\,T}{d}$ and presents, as far as is known to-day, the only source of cohesion in a clean, wet sand. Fig. 16 shows graphically the manner in which surface tension acts. The relation between the increase of ΣF and of d can be expressed by a symbolic term similar to Equation (2). Therefore, for a certain proportion of water, the cohesion presents a maximum, which agrees with the observed facts. The same is true if the volume of voids of the sand is reduced by compacting, while the moisture remains unchanged. In this case, it depends on the moisture, whether compacting causes an increase or a decrease of cohesion. The surface tension of the water continues to act after the motion of the sand has started, because the grains remain in contact with each other. Hence, if the cohesion decreases while the moisture remains unchanged, it is known that the total area, ΣF, of the surface of contact between the sand grains has become smaller; the structure was loosened, which, in turn, substantially affected the internal resistance. All these complications, however, mean very little, if compared with the fact that Mr. Feld's "internal resistance" includes cohesion in terms of an angle. Even more difficult is it for the writer to follow the author through the theoretical part of his paper. All the empirical

arguments which lead to the rules for the application of the wedge theory in the computation of the earth pressure are based on the assumption, that the "internal resistance" is identical with the internal resistance of the back-filling. In case the identity does not exist, the arguments lose their validity.

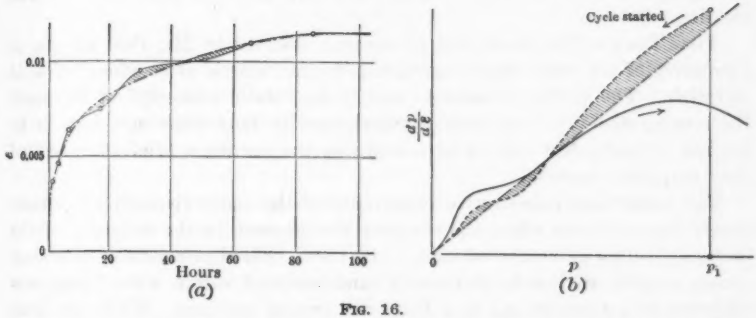


FIG. 16.

The writer cannot possibly accept any of Mr. Feld's suggestions defining or determining internal resistances. The clear-cut distinction between resistances of Types (*a*) and (*b*) stands as firm as before. All attempts to measure the "internal friction" directly by experiments made independently of the earth-pressure test will furnish misleading results, simply because the conditions within the back-filling cannot possibly be imitated reliably. They are independent of Maxwell's important "historical element". The values of ϕ_0, ϕ_1, and ϕ_2, must be deduced indirectly from the earth pressure acting against a vertical retaining wall, before any yield has occurred, immediately after the first slip, and at the final breakdown of the equilibrium. Experiments on a rather large scale, with sands of different kinds and densities, should furnish the limits within which the values of the three angles may possibly vary. These tests should be made only with cohesionless materials, because, due to cohesion, the ratio, $k = \frac{H}{\frac{y\,h^2}{2}}$, becomes different from what Mr. Feld assumes as a function of height. That makes it still more difficult to establish simple relations between theory, experiment, and practice.

Effect of Static Load.—Mr. Feld points out: "There is an appreciable increase in pressure due to loads not on the wedge of rupture. The increase is much more rapid, however, as the load moves on the theoretical wedge". It may be remembered that the back-filling was in an advanced stage of the first phase. According to the writer's conceptions, this fact would be less obvious in an earlier stage of the first phase; and, during the second phase, it would come into still greater prominence.

The changed side pressure continued after the load was removed. This fact is by no means due to lack of elastic behavior by the back-filling, but to the fact that the load has exceeded the elastic limit of the ground. The elastic rebound which occurs after removing a fairly concentrated load of

100 lb., is a small fraction of the permanent deformation; but, on the other hand, the permanent deformations are strictly confined to the pressure bulb. A movement of the wall of 0.006 cm., occurring after the load was removed, would have re-established the original side pressure. Hence, the change in density due to the load was not more important than that caused by the wall yielding 0.006 cm.

Time Tests.—The interesting phenomena observed by Mr. Feld are due to a property of the sand which corresponds to the "elastic after effect" of solid materials. The writer proposes to call it the "static viscosity" of the sand. He investigated this phenomenon experimentally four years ago, but, as he has not yet published any report concerning the results, a brief statement of them may be of interest.

The nature and comparative importance of the static viscosity come most clearly into evidence when one observes the decrease in the intensity of the internal stresses in a mass of sand, while the outside dimensions of this mass remain strictly unaltered. A mass of sand, confined within a steel ring, was subjected to a pressure, p_1, in a Universal testing machine. While the head of the machine remained in position, the pressure of the sand acting against it decreased and asymptotically approached a value of p_1'.

Let,

p = the pressure acting against the piston at a time, t, after the pressure, p_1, was applied;

p_1 = the pressure which corresponds to a time, $t = \infty$; and,

$\Delta p = p - p_1'$.

The decrease, $\frac{d p}{d t}$, in pressure was found to follow the law:

$$-\frac{d p}{d t} = \left(\frac{\Delta p}{\alpha}\right)^3 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Characteristic values of Δp_1 and α are presented in Table 38. For sands, α increases almost in direct proportion with p_1; for clay cubes, it was found that it increases in direct proportion with the capillary pressure, p_k, under which the clay stands.

As the decrease in pressure takes place while the outside dimensions of the body remain practically unaltered, it is evident that inter-granular movements of decreasing intensity occur within the sand. The physical cause is due to the fact that the grains cannot adjust themselves to the altered statical conditions as rapidly as the pressure, p_1, is applied. As a consequence, unbalanced frictional resistances appear, the mutual compensation of which occurs but gradually. The cumulated effect thereof consists in a decrease of the total internal pressure.

In the case of the back-filling of a retaining wall, the pressure, p_1, which is represented for each horizontal interface by the weight of the overlying mass of sand, remains constant, while the settlement proceeds.

Let,

ε_1 = the proportionate compression immediately after the pressure, p_1, was applied;

ε = the same, but at a time, t, following that event; and,

ε'_1 = the proportionate compression which corresponds to a time, $t = \infty$, and $\Delta\,\varepsilon = \varepsilon - \varepsilon'_1$.

The increase in compression was found to follow the law:

$$\frac{d\,\varepsilon}{d\,t} = \left(\frac{\Delta\,\varepsilon}{\beta}\right)^3 \qquad (5)$$

The relation is very similar to that which was observed by Michelson* for solid bodies, although he has preferred to express the results of his measurements by a rather complicated transcendental function. The effect of a rapid application of a pressure on the surface of a mass of sand is, in harmony with what Michelson has found for solid bodies, a threefold one: (*a*) Rapid elastic yield; (*b*) elastic-viscous displacement; and (*c*) viscous displacement. There is in addition (*d*), the lost motion or the permanent deformation, which occurs in connection with Types (*b*) and (*c*), even in case the stress acts for a very short time. The displacements by Equation (5) are of Type (*c*).

TABLE 38.

No.	Kind of material.	p_1, in kilogrammes per square centimeter.	a_1, in kilogrammes-centimeter-minutes.	$\Delta\,p_1$, in kilogrammes per square centimeter.	$\frac{a}{p_1}$	$\frac{\Delta\,p_1}{p_1}$
1	Pulverized quartz, very loose	16.9	7.51	3.86	0.445	0.228
		27.11	8.75	4.42	0.323	0.163
2	Pulverized quartz, compacted	30.0	2.75	3.2	0.0915	0.106
		61.05	5.69	7.2	0.092	0.118
		120.45	7.33	12.52	0.059	0.101
3	River sand with rough grains, compacted	30.0	3.46	4.05	0.115	0.135
4	Sand with silt and clay, compacted	29.5	2.60	3.02	0.088	0.102
		60.5	4.67	6.90	0.077	0.114
5	Dry pulverized yellow loam, compacted	30.4	2 38	2.66	0.0782	0.0875
		60.0	6.80	7.15	0.113	0.119

p_1 denotes the pressure acting at the outset of the time test, p_1' the pressure acting after a time $t = \infty$, and $\Delta\,p_1 = p_1 - p_1'$.

If the observations of the increase in compression under a constant load cover a period of several days, the curve connecting the measured points with each other deviates from the theoretical line in much the same way as damped oscillations would be shown in a diagram. (Fig. 17 (*a*).) Another case of such "damped oscillations" was noticed by the writer when he investigated the relation existing between the change, $d\,p$, in pressure and the corresponding change, $d\,\varepsilon$, in compression for a complete cycle of loading (gradually reducing a pressure, p_1, to zero and applying it again). (See Fig. 17 (*b*).) In loams and clays this property is developed still more characteristically.

* "The Laws of Elastic-Viscous Flow", *Proceedings*, National Academy of Sciences, May 15, 1917.

These phenomena of damped oscillations are apparently a consequence of the granular structure of the sand, and give a clear idea of how the gradual compensation of the unbalanced frictional resistance occurs: The internal stability of the structure of a sand obviously depends on the frictional resistances remaining unaltered, because each grain is kept in its position by frictional resistance. If this resistance diminishes gradually, on account of the mutual compensation of its unbalanced part, the structure becomes more and more unstable, until an adjustment of the grains to the altered conditions becomes imperative. This adjustment represents the period of accelerated settlement. After it is accomplished, it again requires a certain amount of decrease in internal friction, until the lack of stability becomes important enough to require a re-adjustment.

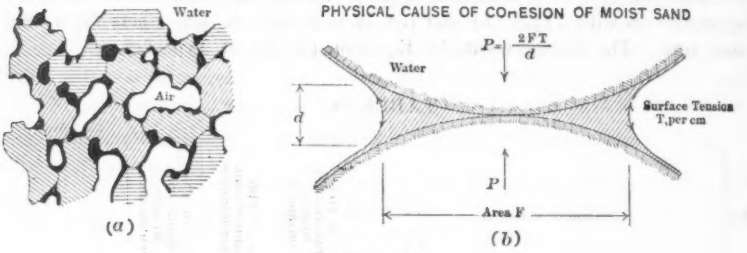


FIG. 17.

The preceding remarks concern the relation between pressure and settlement only. The author's observations have shown that the periodicity in settlement affects the intensity of the side pressure as well. The oscillations observed by Mr. Feld are characterized by a remarkable lack of uniformity. This fact confirms the writer in his conviction that, within the wedge of rupture, the static conditions are far more complicated than is generally assumed. The bottom part of the wedge comes into the second phase while the top part is still in an initial stage of the first one. This state of affairs will undoubtedly be seen as soon as an experimenter undertakes to investigate the distribution of the earth pressure over the back of the wall, by using pressure cells or similar devices built into the wall.

The Vertical Component of the Earth Pressure.—The results of the Cincinnati tests leave little doubt concerning the fact that the friction between the back-filling and the back of the wall is fully active. That seems to be due to the settlement of the back-filling, which has the same effect as an upward movement of the wall of equal importance. Therein lies an essential difference between the results obtained with a small and with a large apparatus, because the settlements in fills, with a height of only a few inches are apparently not sufficient to develop the full frictional resistance. With fills 6 to 9 in. in height, Mr. Feld apparently obtained values about the same as the writer did when experimenting with his small apparatus. (Table 4, Column 9.) Mr. Feld explains the low values by the wall having been dry during this

stage, but the writer has obtained, for the angle of friction between sand and dry timber, a value of 34 degrees. Hence, it is difficult to accept Mr. Feld's argument.

Less comprehensible is the contradiction between the results obtained by Mr. Feld and the results of the very careful measurements made in 1891 by Mr. A. Donath* in the Polytechnikum in Berlin, with fills 24 in. high. In these tests, the wall was suspended and counterbalanced by an exceedingly sensitive device, whereby a column of mercury was used for the purpose of detecting any movements. Nevertheless, no movement occurred, nor was the vertical component found to be greater than zero. Mr. Feld obtained an angle, $\phi' = 24°$, for fills 24 in. high.

It remains to be determined whether the friction acts during an early state of the first phase or during the second one (after the first slip has occurred) as well. With the Cincinnati apparatus, such an investigation cannot be made, because the trig loops are not removable. However, there is another question, of the utmost practical importance, that could be answered by a few supplementary tests: If the angle of friction between the back-filling and the back of the wall increases from zero to ϕ', the intensity of the horizontal component should decrease, according to the wedge theory, from

$$H = \frac{\gamma h^2}{2}\left(\frac{\cos\phi}{1+\sin\phi}\right)^2 \text{ to } H' = \frac{\gamma h^2}{2}\left(\frac{\cos\phi}{1+\sqrt{\frac{\sin(\phi+\phi')\sin\phi}{\cos\phi'}}}\right)^2 \quad \ldots\ldots(6)$$

provided the angle, ϕ, remains unchanged. For $\phi' = 32°$ and $\phi = 37°$, the difference between H and H' should amount to 0.28 H'. Hence, if one succeeds in suppressing the vertical component of the earth pressure, the horizontal component should increase by fully 28 per cent. The test could be made either by lowering the vertical contacts, after the horizontal component had been measured, until the vertical component becomes equal to zero, or by two independent experiments, whereby, during the second one, only the dead weight of the wall should be counterbalanced. In addition, one could investigate the effect of the friction component acting in the opposite manner.

The results of this supplementary test would be important as an argument in any future discussion of the wedge theory, and could lead to a fundamental change in the conception regarding the equilibrium of the wedge of rupture. Remembering his own observations, the writer believes that this difference will not be as important as required by theory—provided it exists at all. The experiments should be made with dry sand.

Design of Apparatus.—The measuring device with platform scales gave excellent results, and will undoubtedly become a standard feature in similar investigations. Unfortunately, in the Cincinnati arrangement, the advantages offered by the sensitive measuring device have been more than counterbalanced by the excessive flexibility of the wall, combined with the impossibility of extending investigations at least beyond the point of the "first slip". Another objectionable feature of the arrangement was the small width of the

* "Untersuchungen über den Erddruck auf Stützmauern", *Zeitschrift für Bauwesen*, 1891, p. 491.

bin as compared with the height of the fill. The sliding device used by Mr. Feld was not capable of satisfactorily compensating for the effect of the narrowness of the bin. That would have been the case only if, within the wedge of rupture, the motion of the sand grains was strictly parallel to one another; but, in reality, the lateral expansion involves intergranular displacements deviating to each side from the general trend of the movement. Mr. Feld observed that the decrease in resulting pressure was "not very large", due to fixing the sliding wings in their position, and there might be a considerable difference between the pressure acting per unit of length of a gate, according to whether that length was equal to the height of the fill or to twice that height. This assumption is suggested by the results of small-scale experiments. The shape of the curve Fig. 15 (*b*), is an argument neither in favor of nor against the assumption being correct, because the intensity of the pressure was influenced by the deflection of the wall. This deflection increased with the height of the fill.

Conclusions

1.—The experiments were supposed to furnish the intensity of the earth pressure acting against an ordinary retaining wall, which yields throughout a distance of not more than $\frac{1}{3\,000}$ to $\frac{1}{18\,000}$ in., that is, which is practically unyielding. As the deflection of the model retaining wall (gate) amounted to many times the distance over which the gate was allowed to yield, the purpose was not accomplished, and the results do not interfere with the writer's conception of the "first phase".

2.—The deformations of the gate were so large, in comparison with the movements required to produce important changes in pressure, that the back-filling as a whole was in an advanced stage of the first phase (Fig. 15), not far from the first slip. As the deflections of the gate evidently decreased from the center of the gate toward the three horizontal contacts, the arch effect was inevitable, involving unequal distribution of the pressures along horizontal lines. This arch effect is absent in the back-filling of an ordinary retaining wall, hence, no conclusions can be drawn from the results of the test, until the deflections of the gate and the distribution of the pressures have been investigated. It is recommended that both the calculated and the measured deflections of the gate be published.

3.—The retarding effect of the side-walls has not been completely excluded. Thus, the results were influenced by another factor of unknown importance.

4.—The method of measuring the internal resistance with a sliding box furnishes values which have nothing in common with the internal friction within the back-filling, because the latter is a function of the amount of yield of the wall, and the former possesses a definite value. Mr. Feld's definition of the angle of internal resistance involves a contradiction, because the cohesion, being equal to a constant per unit of surface, cannot be expressed in terms of an angle.

5.—As the cohesion appears in the paper in terms of an angle, the reader is unable to ascertain what the cohesion per unit of surface actually has been and is unable to make an independent analysis of the results of the tests.

6.—Each report on earth-pressure tests should contain all the data required for judging the state of density and internal stability of the back-filling. If it is found, for instance, that the back-filling weighed 96 lb. per cu. ft. and contained 3.5% of moisture, one does not even know the volume of voids, unless the average specific gravity of the grains is given; but, even with the volume of voids given, it is not possible to judge the back-filling from a static point of view, because certain sands have 46% of voids, even when fairly compact, and others must be considered as loose, depending on the mechanical composition of the sand and the quality of the grains. Therefore, besides the results of mechanical analysis and the actual volume of voids, there should be stated the volume of voids of the sand, both if compacted by ramming and shaking under standard conditions and if loosely poured into a vessel. The relation between the actual volume of voids and the two other values has an important bearing on both the ultimate resistance and the effect of the wall movements in an advanced stage of the experiment. Only when these data are presented, can the properties of the back-filling be said to be known to the outsider as well as the experimenter.

7.—Internal resistance and internal friction, as defined and determined by Mr. Feld, cannot be considered as being physical characteristics of the back-filling. Concerning this fundamental point, no compromise whatsoever is possible between Mr. Feld's ideas and the writer's views, and either one of these two conceptions must be discredited. According to Mr. Feld, the coefficient of internal friction is independent of the movement of the wall, and, therefore, has a definite value. According to the writer, it depends to a great degree on the movement of the wall, as shown in Fig. 15 (*a*). Its value, therefore, is determined merely inasmuch as it is confined to the interval between two limiting values, $\tan \phi_0$ and ϕ_2, whereby ϕ_2 is more than twice as large as ϕ_0. In addition to this indeterminate internal friction, acting throughout the wedge, there is another, $\tan \phi_1$, opposing separation along a plane, which, however, determines only the side pressure at which the first slip (s_1, in Fig. 15 (*a*)) occurs; but, as the Cincinnati tests did not involve any slip (Fig. 15 (*b*)), on account of the restricted movement of the wall, it is impossible for the resistance, $\tan \phi_1$, to enter into the discussion of these experiments.

The term "angle of repose" has no meaning, unless the material is free of cohesion. The steepest possible slope, in deposits consisting of materials with cohesion, depends on the height of the sloped deposit.

8.—No theoretical conclusions can be drawn from the results of the Cincinnati tests until the open question concerning internal resistance (mentioned in the preceding Paragraph 7) has been decided. Mr. Feld's suggestion, to exclude the wall friction from the formulas for computing the earth pressure against vertical retaining walls and to consider it in every other case, seems to be rather arbitrary, and does not represent a satisfactory solution. The writer has also specified in detail his doubts concerning the validity of the numerical arguments presented by Mr. Feld in favor of his suggestion.

The hopeless confusion which continues to prevail in earth-pressure literature is due to the ignorance of the fact that the internal friction is a function

of the amount of lateral expansion of the back-filling, and that it varies accordingly between very wide limits. As explained in detail by the writer, even Mr. Feld's new distinction between internal resistance and internal friction seems hardly likely to clear up the obscurity, in spite of its being the outcome of a study of 650 papers on earth-pressure theory. The close agreement of the results of independent experimental work, mentioned by Mr. Feld on page 1505, is merely due to the fact that the measurements are almost inevitably made toward the end of the first phase, where the curve turns abruptly downward, because a comparatively small movement or deflection of the wall is sufficient to produce this state, indicated in Fig. 15 (*a*) by the cross-shaded area. The Cincinnati tests, as has been shown, make no exception. Experiments on a large scale, however, never extend beyond the first slip, on account of the difficulty of preventing the sand from flowing out. Hence, all the measurements are confined to a very limited range at the end of the first phase only, which leads to the erroneous opinion that the problem is statically determinate. This error must disappear, and there is no chance of arriving at a final agreement in the earth-pressure question until experimenters fully realize the utterly variable character of the internal friction and learn to distinguish sharply between friction in the space and friction along a plane, which are two fundamentally different things. Future experimenters should proceed with the intention of investigating, for different materials and under different conditions, the limits, ϕ_0 and ϕ_2, between which the angle of internal friction may possibly vary. Especial efforts should be made to determine the angle of maximum resistance, ϕ_2, because this angle determines the utmost strain which the sand can stand. No data are available on the value of this important angle, except the few figures obtained by the writer with very small apparatus. The ultimate bearing capacity of the sand, as well as the limiting state of every other earth-pressure phenomenon which does not involve a normal slip, depends on the value of this angle. Hence, the problem consists in constructing a large size apparatus such as to permit the investigation of the whole range of pressure included between the limits, $\frac{y\,h^2}{2}\,t\,g^2\,\frac{90\,-\,\phi_0}{2}$, and $\frac{y\,h^2}{2}\,t\,g\,\frac{90\,-\,\phi_2}{2}$. Until this problem is solved, no test will realize any material progress beyond the limits of what has been already accomplished. This statement is graphically presented in Fig. 15, and the supplementary Cincinnati tests suggested by the writer, will prove its truth.

The writer wishes to express his admiration for the remarkable precision with which Mr. Feld made his observations, but he cannot agree with his attitude toward experiments with miniature apparatus. Without the experience acquired while patiently experimenting with small apparatus, the writer would have been unable to assume toward the Cincinnati results as clear-cut an attitude as he has. Although his experiments furnished no conclusive numerical values, they made the writer thoroughly familiar with the whims of soils. In connection with soil mechanics, the miniature tests play the same part as the reconnaissance does in the field of railroad location: They require the same mature judgment and experience, and offer to the engineer ample

opportunity for committing serious blunders. Nevertheless, no one would dare to undertake a final location without the preliminary investigation.

WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).—The writer has been interested in Mr. Feld's experimental work ever since Professor Braune sent him the original design of the apparatus. This, as finally constructed, seems to have been admirably designed for the ends in view, as it measures accurately, not only the horizontal component of the earth thrust against the retaining board, but likewise its vertical component, whether the board is vertical or inclined (forward or backward), and for an earth surface either horizontal or inclined.

In all cases, the experiments prove that the direction of the earth thrust is always inclined to the normal to the retaining board, or wall, at an angle equal to ϕ', the angle of friction of the earth on the wall. Such experimental evidence definitely settles this controverted question as to the direction of the earth pressure on a retaining board, and is in line with the results of all previous experiments on retaining boards, or walls, of various types.

In times of heavy rains, the backs of actual retaining walls may be lubricated with water, so that ϕ' is considerably diminished, and, in the design, account should be taken of this possible change of direction of the thrust. When the increase of moisture is not excessive, the internal angle of friction is generally increased and the normal component of the thrust is diminished; but it is not on the side of safety to allow for this; hence the writer has recommended, in design, the use of the old normal component for dry earth, but that the thrust be assumed as making an angle with the normal to the wall much less than the angle of friction of the dry earth on masonry.

The author has conducted some experiments on glass and sheet-metal walls, and, from them, draws the conclusion that "the horizontal component of the pressures of any kind of fill on a vertical wall is independent of the nature of the wall surface." This conclusion is drawn from a comparison of Test No. 1 with Tests Nos. 3 and 4, or of Test No. 5 with Test No. 7, and is too sweeping in statement. Thus, the angle of wall friction, ϕ', for the wooden wall, when dry, is given as 30°; when damp, 32° 30′; whereas ϕ' for the glass wall is 28° 10′, and for the sheet-metal wall, 28°; so that the difference in ϕ' is only 2° to 4° 30′. Besides, the percentage of moisture in the fill and the angle of internal friction varied materially for the different cases, so that a direct comparison cannot well be made between them. If it was possible, by some device, to make ϕ' very small for a certain wall and then compare the resulting lateral thrust with that for an ordinary wooden wall for the same ϕ and percentage of moisture and temperature, the results would be more convincing.

From the experiments, the author has reached some valuable conclusions relative to the center of pressure of the earth thrust on a wall. If the earth is in contact with the wall for a height, h, the center of pressure, for all types and kinds of wall, and for all types of loading and fill, is found to lie between 0.33 h and 0.4 h above the base of the wall. This is in agreement with the

* Prof. Emeritus of Math., Univ. of North Carolina, Chapel Hill, N. C.

results of Leygue. For a vertical wall, with the earth surface horizontal or inclining downward ($\epsilon = 0$ or negative), the center of pressure is at $\frac{1}{3}\ h$ from the bottom of the wall; for the earth surface rising from the wall (ϵ positive), the limit, 0.4 h, is approached. From Table 24—tests for a wall having a positive batter—it is seen that, for ϵ negative, the center of pressure is practically at $\frac{1}{3}\ h$; but for ϵ positive the value 0.4 h is approached and even exceeded. For walls with a negative batter (Table 25), the conclusion is drawn that the heights of the centers of pressure vary from 0.35 h to nearly 0.4 h for positive or negative surface slope.

It is well to bear these variations in mind in designing retaining walls. However, as walls are generally designed, either for the earth surface level or sloping indefinitely at the angle of repose, and with a positive batter, for which cases, the theoretical exceeds the experimental thrust considerably, the result will be on the side of safety if the theoretical thrust is considered as acting at the third point, the thrust being computed for a true internal angle of friction, and ignoring the influence of cohesion.

For the usual surcharged wall, with the earth surface rising from it at the angle of repose to a certain height and then continuing in a horizontal direction, the thrust can be found from a graphical construction, and, in fact, the theoretical center of pressure determined.* In view of the experiments, it may be preferred, in the actual design of surcharged walls of the usual kind, to assume the thrust as acting at 0.4 h above the base, as that is certainly on the side of safety.

When cohesion is neglected, the theoretical thrust on a vertical plane—for the case of the earth surface sloping indefinitely upward at the angle, ϕ, of internal friction—is always greater than the actual thrust. This is true because all earth—even sand—is endowed with cohesion, and its influence is to diminish the thrust, this diminution being greater the larger the coefficient of cohesion. Thus, by a construction,† for a wall 4.8 ft. high, the values of the coefficients, K_1 and the approximate positions of the planes of rupture, $A\ C$, Fig. 18, were determined for,

ϕ = angle of internal friction = $\phi' = 33° 41'$,

c = coefficient of cohesion, in pounds per square foot, as given on Fig. 18,

y = weight of earth = 80 lb. per cu. ft., the horizontal component of the thrust being given by the expression, $\frac{1}{2}\ y\ h^2\ K_1$.

The first value of K_1 (in the following tabulation) was found by computation, the other values by construction:

$c = 0$, $K_1 = 0.692$	$c = 10$, $K_1 = 0.366$
$c = 4$, $K_1 = 0.474$	$c = 20$, $K_1 = 0.240$
$c = 6$, $K_1 = 0.430$	$c = 30$, $K_1 = 0.150$

* See the writer's "Earth Pressure, Walls and Bins", (John Wiley and Sons), pp. 66 and 121; or *Transactions*, Am. Soc. C. E., Vol. LXXII (1911), pp. 404–411.

† *Transactions*, Am. Soc. C. E., Vol. LXXII (1911), Fig. 11, p. 416, and Fig. 13, p. 425. The more exact construction of Fig. 25, p. 461, when the coefficient of cohesion is small, gives practically the same results as the simpler construction of Fig. 11, so that the latter was used.

The theoretical plane of rupture approaches indefinitely, as a limit, a position parallel to the surface, when $c = 0$. From Fig. 18 the great influence cohesion has in changing the inclination of the plane of rupture may be noted. The values of K_1 show how rapidly the thrust decreases as c increases, and, for this case, why the actual is always much less than the theoretical thrust when the cohesion is neglected.

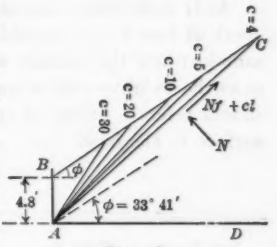


FIG. 18.

As the inclination of the earth surface varies from $\epsilon = +\phi$ to $\epsilon = -\phi$, the plane of rupture varies,* but there is no warrant in assuming, for a vertical wall and $c = 0$, and for all cases of surface slope, that the theoretical plane of rupture bisects the angle between the vertical and a line inclined at the angle, ϕ, to the horizontal. Perhaps the assumption by the author to that effect (page 1474), was not regarded as having a strict theoretical basis.

From the writer's investigations,† it was found "for walls about 6 ft. high, that the earth thrust, neglecting cohesion, was only from 1 to 5% in excess of that for $k = 1$" (that is, 7 lb. per sq. ft.). Hence, walls 6 to 10 ft. high were recommended for future experimentation. It was this suggestion that caused Professor Braune to adopt the 6-ft. retaining board. It is unfortunate, but perhaps unavoidable, that the leading experiments—those on 5 and 6-ft. walls—were made with sand containing the highest percentage of moisture, with a consequent appreciable cohesion. In fact, Leygue found, from his rather restricted experiments, for the pure sand used, that

For dry sand.....	$\phi = 35°$	; $c = 1.47$ lb. per sq. ft.
" wet sand.....	$\phi = 40° 22'$	; $c = 8.28$ " " " "
" very wet sand.....	$\phi = 59° 30'$	; $c = 6.36$ " " " "

As the normal pressures were very small—only 7 to 40 lb. per sq. ft.—they do not cover sufficiently the normal pressures on the planes of rupture for walls 5 or 6 ft. high, which varied from zero at the surface to several hundred pounds at the base. In fact, for $y = 100$ lb. per cu. ft., the vertical pressures on horizontal planes at the base of such walls, were 500 to 600 lb. per sq. ft.

In Tests Nos. 8 to 18 and No. 25, the bank sand was comparatively dry, the moisture varying only from 3.25% to 3.7%, with 3.55% as an average. The angle of internal friction varied from 27° 30′ to 29° 40′, with an average of 28° 24′; and the weight of the sand per cubic foot varied from 93 to 97 lb., with an average of 95.7 lb.

The results for the remaining tests—made generally on the higher walls (5 or 6 ft. high)—are given, in part, in Table 39.

The moisture for Tests Nos. 5, 6, and 7 is not given, but, presumably, it is approximately 9 per cent. This large percentage of moisture, in all these tests except Nos. 3 and 4, involves an appreciable cohesion—probably 8 lb. per sq. ft.

* See the writer's "Earth Pressures", p. 63, for comments as to this variation.

† "Experiments on Retaining Walls and Pressures on Tunnels", *Transactions*, Am. Soc. C. E., Vol. LXXII (1911), p. 422.

As it is doubtless impossible to obtain a homogeneous sand throughout the depth of 5 or 6 ft., it would be interesting to know how the author selected his samples; how the weights and percentages of moisture were determined; and as to the weights used in ascertaining the angles of friction. These quantities should refer to filling of the average depth, as they doubtless vary from the surface to the base.

TABLE 39.

Test No.	Table No.	Percentage of moisture.	Angle of internal friction.	Weight of earth, in pounds per cubic foot.
1	3	9	34°	100
2	4	9	34°	100
19-24	26-32	9	34°	100
3	6	4.9	31° 50′	93
4	7	4.2	34° 50′	94
5	8	....	35° 30′	95
6	9	....	35° 30′	95
7	10	....	32°	94

The author is to be commended for the painstaking care with which the experiments relative to earth pressures have been conducted. They cover a wide range, and, for the object in view, are doubtless the most accurate on record. The results are likewise presented clearly in the numerous tables. However, an obvious criticism is that the author's method of determining the angles of friction, is not sufficiently complete to determine such angles for the conditions of their use. In order to interpret results properly—especially in connection with standard formulas—it is necessary to determine the angle of internal friction, ϕ, and the coefficient of cohesion, c, after the method outlined by the writer in a previous paper,* the method originating with French and English experimenters.

The simple apparatus used by J. H. Griffith, M. Am. Soc. C. E.† in making shear tests on clays, seems to be amply adequate to conduct the tests for determining c and ϕ in sand filling for various depths. Fig. 19 represents this apparatus in principle, without the details. The lower box, without a top, is filled with the earth, which extends into an upper box having neither top nor bottom. The depth of the layer of sand in the upper box should be as little as experience deems practicable, as its horizontal thrust on the vertical sides of that box, causes "wall friction"; so that a certain proportion of the weight of sand above the bottom of the upper box, and of the load placed on it, is carried by the sides of this box and will be transmitted to the rollers of Mr. Griffith's device, not shown in Fig. 19. These rollers are placed between the edges of the boxes, and do not extend into the sand. The weight thus transmitted to the sides of the upper box can be ascertained, perhaps with sufficient accuracy, by measuring the vertical force necessary to draw the upper box upward and away from the sand. It should be very small for the thin lamina of sand contained in the upper box. Above this sand is placed a board (which must

* "Cohesion in Earth", etc., *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), pp. 1315–1341.

† *Proceedings*, Am. Soc. C. E., March, 1922, Papers and Discussions, p. 565, Fig. 7.

be level), on which is placed the weight. This board insures uniform distribution of the pressure.

After deducting the weight transmitted to the sides of the upper box, let P_n represent the remaining weight, in pounds, of sand above the plane of sliding, AB, and the load above it. Also let,

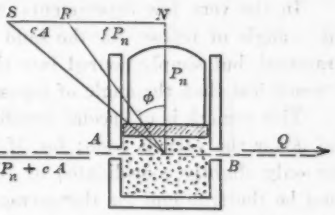


FIG. 19.

A = area of this plane, $A\,B$, in square feet;
ϕ = angle of internal friction, $f = \tan \phi$;
c = cohesion, in pounds per square foot;
Q = horizontal force in pounds, applied to the upper box in the plane $A\,B$, just sufficient to cause sliding.

Then, when motion is impending, by Coulomb's laws, assumed provisionally as true,

$$Q = f P_n + c A = P_n \tan \phi + c A.$$

On dividing both sides of this equation by A, and designating by q the total resistance to sliding, in pounds per square foot of the plane, AB; and by p_n the normal pressure, in pounds per square foot, of this plane, we have,

$$q = f p_n + c = p_n \tan \phi + c.$$

The extended experiments of Bell on clays of a certain kind, and of MM. Jacquinot and Frontard on earth from a reservoir dam,* practically sustain the laws of Coulomb, but sufficient experiments on sand are lacking. A large field is open to the experimenter along these lines, for it is necessary to include every kind of earth in the experimentation.

In the apparatus represented in outline by Fig. 19, different weights are used, the simultaneous values of p_n and q_n are plotted, and an average straight line is drawn through or near the points. The angle that this straight line makes with the axis of p_n is ϕ, the angle of internal friction, and the intercept on the axis of q is c, the coefficient of cohesion.

The results for the first set of experiments on the earthern dam by MM. Jacquinot and Frontard, are shown in Fig. 20, leading to the values $\phi = 8° 15'$, and $c = 385$ lb. per sq. ft. The consolidated earth of this dam, tested: 60% clay, 32% silica as an impalpable dust, and 8% silicious sand.

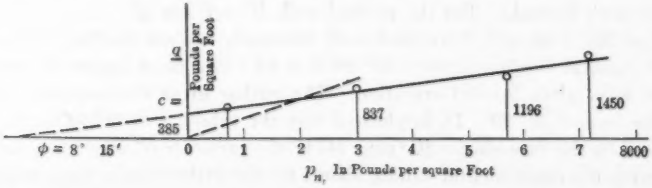


FIG. 20.

* Given in some detail by the writer in *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), pp. 1320–1324, and pp. 1332–1335.

In the very few experiments on "perfectly dry sand", Bell found, $c = 0$ $\phi =$ angle of repose, for the sand well rammed; but when the sand was not rammed, but simply poured into the cylinder and shaken, ϕ was found to be "much less than the angle of repose".

This remark is of special significance as regards the author's determination of ϕ for the various tests; for, if the sand in the box was simply poured in, or only slightly consolidated by light weights, the resulting value of ϕ may not be the true one for the average depth of fill. The state of consolidation should be as nearly as possible what it is in the filling behind the retaining board. Thus, for heights of earth next the wall of 5 or 6 ft., the weights P_n should vary from a minimum sufficient to consolidate, to a maximum of about 500 to 600 lb. per sq. ft. of the area, A; for it is known that c and ϕ vary with the density of the earth, and c, particularly, can be altered very much by ramming.

The full significance of the pressure to be used can be shown by Fig. 18. If AC is a plane of rupture for a certain c and ϕ, and N is the normal reaction of this plane, then the resistance to sliding down it is, $N f + c l = N \tan \phi + c l$, where $l = AC$. Here, the f and c actually exerted is the average for the length, AC, corresponding, roughly, to the consolidation of the earth at the average depth.

The writer is satisfied that the determination of c and ϕ by the method just outlined, will give results nearer the truth than the author's method. Whether the new results will tend to harmonize theory and experiment remains to be seen. It will certainly prove unfortunate if they do not. The "angle of resistance", as defined by the author, does not seem to be very definite, as it probably depends on the load placed on the sand in the apparatus used. Thus, the dotted line in Fig. 20, which makes "the angle of resistance" with the axis of p_n and passes through the origin, will attain the same value of q as the true line, $q = p_n \tan \phi + c$, only where it meets it, or only for one value of p_n.*

To illustrate how changes in ϕ, the angle of internal friction, will affect the lateral thrust, consider Test No. 1. The author gives $\phi = 34°$, $\phi' = 32° 30'$. If it is assumed that $\phi = 30°$, $\phi' = 32° 30'$, $c = 0$, then, from the general wedge theory, $n = 0.7252$ and $H' = 0.252$, and this value will give, practically, the middle curve of Fig. 3, which agrees so well with the experiments. The thrust, in the "general wedge theory", is always assumed to make the angle, ϕ', with the normal to the wall, and has the value, $E = \frac{1}{2} y h^2 C$, where C is given by the author's formula. For the vertical wall, $H' = C \cos \phi'$.

Test No. 5 on a 5-ft. vertical wall, the earth surface making angles with the horizontal varying from $-33° 40'$ to $+34°$, is a most important one, and it has been given special attention. The author gives the angle of internal friction $= \phi = 35° 30'$. If, in place of this, the values, $\phi = 32° 30' = \phi'$, $c = 0$, are used in the formula for H' (page 1474), the curve for H', Fig. 21, agrees very well with the experimental values, shown by the little crosses, for all negative slopes, and for all positive surface slopes up to near 30 degrees. It has been seen,

* See further comments on this subject by the writer in *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1334 and 1339.

in connection with Fig. 18, why the theoretical thrust for $c = 0$, becomes increasingly too great when ϵ is near ϕ for the actual earth endowed with some cohesion.

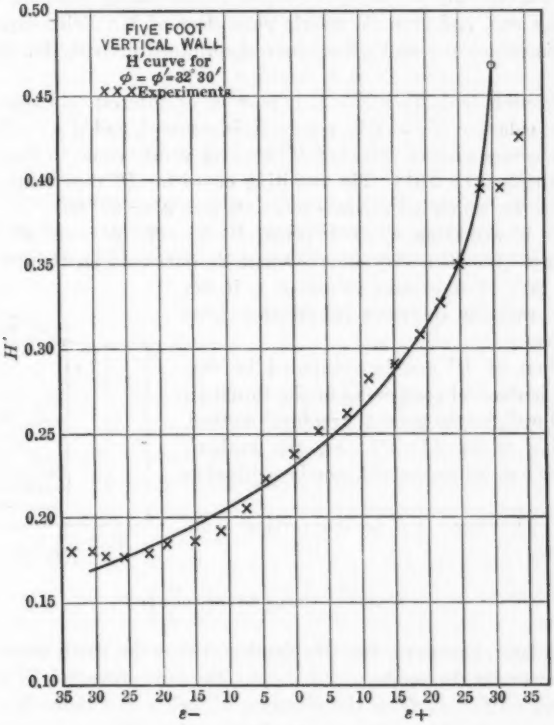


FIG. 21.

It may be well to remark that all the coefficients in Table 8, Column 14, are really values of C. They should be multiplied by $\cos \phi' = \cos\ 32°\ 30' = 0.843$ to give H'; which will cause a considerable drop in the corresponding curve in Fig. 4. In fact, from general theory, for ϵ positive, it should lie below the Rankine curve when $\phi' > \epsilon$.

In practical design, it is desirable to use formulas for earth thrust that do not involve cohesion, and it is seen, from the illustrations just given, that safe results can be secured by a proper selection of ϕ and the use of one general formula for all cases. If the values of c and ϕ are determined by the method illustrated by Figs. 19 and 20; then, on using the general formula with this value of ϕ, and ignoring c, the resulting figure should be in excess of the true thrust.

It has long been the writer's opinion that the proper interpretation of experiments on earth pressures can only be made by aid of the theory of

coherent earth—that is, earth having both friction and cohesion. On that account, in his book on "Earth Pressures",* he has given the theory in full, so that it can be used in discussions, whatever the character of the filling—whether sand, ordinary or clayey earths, earth thoroughly rammed or consolidated in any way, and even the nearly pure clays of Mr. Bell's experiments—provided the values of c and ϕ have been determined properly for the earth in question.

In connection with Test No. 5, it may be of interest to know that, if a coefficient of cohesion of $c = 6$ lb. per sq. ft. is assumed, and if $\phi = \phi' = 32° 30'$ is used, the corresponding values of H' are very nearly equal to those given by Fig. 21 multiplied by 0.914. The resulting curve for H' thus drops below the curve of Fig. 21, which corresponds to $c = 0$, $\phi = \phi' = 32° 30'$.

However, by assuming, $c = 6$ lb. per sq. ft., $\phi = 30°$, $\phi' = 32° 30'$, the corresponding curve for H' nearly coincides with the curve of Fig. 21 from $\epsilon = -35°$ to $\epsilon = +20°$. For greater values of ϵ, it lies above and gradually increases its distance from the old curve.

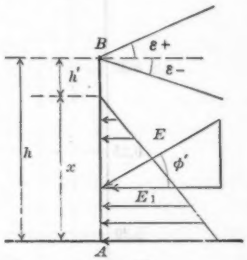


FIG. 22.

The values of H' can be computed by first finding the horizontal component of the thrust on the vertical wall, by the general graphical method, and dividing it by $(\frac{1}{2} y h^2)$; or, the author's formulas for non-coherent earth, can be utilized as follows:

In Fig. 22, where $A B$ is the vertical wall, first compute,

$$h' = \frac{2c}{y} \tan\left(45° + \frac{\phi}{2}\right).$$

There is no lateral pressure for this depth; so that the earth exerts pressure on the wall only for the height, $x = h - h'$. The unit pressure will be assumed as increasing uniformly from the distance, h', below B (where it is zero) to the base of the wall. The actual curve, limiting the ordinates representing the horizontal components of the unit pressures, is concave upward. This curvature is ordinarily very slight, but it becomes very pronounced for $\varepsilon \geq \phi$. However, for any negative values of ϵ and for positive values up to about, $(\phi - 3)$ degrees, the assumption is practically exact, and the horizontal component, $E_1 = E \cos \phi'$ of the thrust E, that makes the angle, ϕ', with the normal to $A B$, can be computed very nearly from the formula.

$$E_1 = E \cos \phi' = (\tfrac{1}{2} y x^2) H',$$

where H' is given by the author's formula for a vertical wall.†

* The general graphical method follows a general discussion, wherein Mohr's circular diagram of stress is utilized, especially in deducing formulas, and in finding pressures and the shape of curves of rupture. In *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), pp. 207–216, a pure analytical discussion is given. The theory of coherent earth was given in this complete form so that it could be at once utilized when the proper experiments were available.

† This method is only applicable to vertical walls. For battered walls, it is very far from the truth, as shown in *Transactions*, Am. Soc. C. E., Vol. LXXII (1911), Table 6, p. 465. For such walls, the graphical method should be used.

Thus, for $c = 6$ lb. per sq. ft., $\phi = 30°$, $\phi' = 32° 30'$, $y = 95$ lb. per cu. ft., $h = 5$ ft., $\epsilon = + 20°$, $\alpha = 0$, it is found, by the formulas, that $h' = 0.22$, $x = 4.78$, $n = 0.441$, $H' = 0.361$; hence, $E_1 = 0.361\ (\frac{1}{2}\, y\, x^2)$.

For a pressure increasing downward uniformly from B, for the height, h,

$$E_1 = H_5'\ (\tfrac{1}{2}\, y\, h^2)$$

where H_5' is the coefficient corresponding to H', but for a height $h = 5$, to agree with the author's notation. On placing these two values of E_1 equal and solving, we derive,

$$H_5' = \left(\frac{4.78}{5}\right)^2 H' = 0.914\ H'.$$

This relation holds for any surface slope. For the case in question, where $H' = 0.361$, $H_5' = 0.330$, which exceeds the value found by the general graphical construction by 2%, and also exceeds somewhat the value given by experiment, Fig. 4.

For $\epsilon = + 27° 30'$, $\phi = 30°$, $\phi' = 32° 30'$, $c = 6$, $y = 95$, $h = 5$, $\alpha = 0$, the formula approximation gives, $H_5' = 0.455$, and the graphical method, 0.435, or 4.6% difference.

The results for battered walls are shown in Figs. 5 and 6. Here, the "moisture content" was only 3.25 to 3.7%, and the cohesion should be less than in Test No. 5. The value of ϕ used in the formulas varied from 29° 40′ to 32°. As the computed thrusts were generally in excess of the experimental values, it will be found, by assuming $\phi = 30°$ $c = 0$ in the general formula, that safe results will be secured.

Attention is called to the curves of Fig. 5, styled: "Wedge Theory—Resultant at ϕ", the earth surface being horizontal. For negative batter, the curve fits very closely that for the experimental results; but, for positive batter, there is quite a difference, the experimental curve likewise having a decided elbow. It is a pity that observations, for this rather dry sand, had not been made for a vertical wall, in order to complete the series. The remaining tests on the influence of rolling loads, temperature, etc., are discussed interestingly in the text.

The comments of the author with respect to some "previous experiments" are timely. When an experimenter finds the angle of friction greater than the angle of repose, it at once condemns his results and shows that he is not well grounded in theory. Experiments on small models should be banned, and where experiments are made with ordinary earth, clayey earth, or tamped earth, any attempted interpretation of results which does not involve the theory of coherent earth is simply farcical.

From all the author's experiments on the bank sand filling, it has been shown by the writer that practical results, on the side of safety, can be attained by assuming $c = 0$ and the angle of internal friction, $\phi = 30°$; the angle, ϕ', being determined by experiment, and the thrusts being computed by one formula—resulting from the wedge theory in which the earth thrust is sup-

posed to make the angle, ϕ', with the normal to the wall.* This value, $\phi = 30°$, is in quite common use for ordinary earth, where the excess thrust is desirable, particularly when it is taken to act at $\frac{1}{3}\,h$ above the base of the wall.

In conclusion, the writer feels that the thanks of the Profession are due the author for the painstaking labor involved in conducting the best tests on a large model up to date. It is to be hoped that the work will be continued.

MILO S. KETCHUM,† M. AM. SOC. C. E. (by letter).—This paper is a valuable contribution to the knowledge of the action of granular materials. In common with most experiments to determine the pressure of such materials, the results are subject to the following limitations:

1.—The width of the bin, 5 ft., was not sufficient to prevent arching in the material, and end constraint.

2.—The pressure-measuring apparatus permitted the material to move forward, thus causing friction to be exerted on the pressure-measuring surface.

The pressures obtained, therefore, are those due to a granular material in a bin, and are not directly applicable to a rigid retaining wall which sustains the pressure of a granular mass. The agreement of the theoretical pressures on a vertical surface with a horizontal surcharge, when calculated by Coulomb's and Rankine's theories, shows that the friction on the pressure-measuring surface did not affect the normal pressure; also, for a vertical wall with a positive inclined surcharge, the Coulomb theory, with a normal component, checked very closely with the experimental values. (See Fig. 4, and, also, Conclusion (*a*) on page 1474).

The material sustained by a retaining wall is in static equilibrium, whereas that in a bin is sustained partly by the bottom and partly by friction on the sides of the bin. To obtain the active pressures in a granular mass, due to the conditions assumed by Rankine, the pressure-measuring surface must be sufficiently rigid to prevent movement of that mass. The pressure-measuring apparatus described in the writer's book‡ is the only one known to him that is not subject to very serious criticism. In it, the pressures are determined by measuring the variation in an electric circuit passing through carbon plates.

It is unfortunate that the author was not more familiar with Rankine's theory of stresses in a granular mass. He has erroneously assumed that the Rankine theory is limited to a vertical wall: (1) with a horizontal surcharge; and (2) with a positive sloping surcharge. Rankine's theory of conjugate pressures in a granular mass leads to the ellipse of stress, which can be applied to vertical walls, walls leaning forward or backward, and with a plane surcharge either level, or sloping, with a positive or a negative surcharge. The stresses in a granular mass under normal conditions, as in the tests made by the author, follow two laws: (*a*) There can be no active upward component in a

* The formula is the first given on p. 1450.

† Dean, Coll. of Eng., and Director, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

‡ "The Design of Walls, Bins, and Grain Elevators," Third Edition, p. 352.

granular mass; and (*b*) the maximum obliquity of the active stresses in the granular mass is equal to the angle of internal friction. The writer has shown* that Rankine's theory, when properly interpreted, applies to walls in all positions.

The author is in error in stating, "The Rankine theory would give an upward component for all the cases of negative ϵ." He is also in error in his statement: "The Rankine theory, as originally given, does not apply to a sloped wall." It is unfortunate that Mr. Feld has not calculated the pressures for the inclined walls by using Rankine's ellipse of stress.

It is interesting to note that the author was not able to discover the plane of rupture. A study of Rankine's theory will show that the plane of rupture is merely a convenient fiction, and does not exist in a granular mass in static equilibrium. When the mass does break, the surface of the break is not a plane, but a curved surface. The so-called plane of rupture bisects the angle between the vertical and the plane of internal friction only for a vertical wall with a horizontal surcharge, with the resultant pressure normal to the wall, as will be given by Rankine. The author is in error in his statement (page 1476), that, for a vertical wall, the plane of rupture in general bisects the angle between the vertical wall and the plane of internal friction.

After careful study of the tests, the writer is more firmly convinced than ever that Rankine's theory of conjugate pressures is the most satisfactory and reliable for calculating the pressures on retaining walls.

HENRY GOLDMARK,† M. AM. SOC. C. E.—Experimental determinations of earth pressure, on a fairly large scale, have been relatively few, so that the author's numerous and accurate measurements and his résumé of earlier tests are valuable contributions to the subject. They show that, for granular materials at least, the commonly accepted theory will give results approximately correct if proper values of the coefficients are used.

In view of the various soils found in practice, it is to be hoped that similar tests may soon be made with many different classes of materials and in the same thorough manner. In addition to such careful laboratory work, it is desirable, in the speaker's opinion, to study the nature of the soil carefully during actual construction and to make field tests whenever practicable. Such experiments are a decided help toward making correct designs, besides leading to a more complete understanding of the entire subject.

It seems worth while, therefore, to place on record an account of some tests recently made during the construction of the new ship lock in the Harbor of New Orleans, La., an important work with which the speaker was connected. This lock, built under the direction of George W. Goethals and Company as Consulting Engineers, is 75 ft. wide in the clear and has a usable length of 600 ft. The depth of water on the sill is 31 ft. and the lift 20 ft., so that the walls are 55 ft. high above the floor. The soil at the lock site is not well suited for supporting heavy loads, as it consists of alternate layers of rather soft clay and sand, the latter saturated with water and under

* "The Design of Walls, Bins, and Grain Elevators."

† Cons. Engr., New York City.

considerable hydrostatic pressure. For this reason, reinforced concrete was used throughout in order to reduce the weight. It was necessary, therefore, to unwater the lock-pit entirely and to keep it dry until the masonry was completed. The difficult work of unwatering, which was carried out successfully by a novel method devised by George M. Wells, M. Am. Soc. C. E., who represented the Consulting Engineers in the work, has been fully described in a recent paper.*

The entire lock-pit was first excavated by suction dredges and subsequently unwatered, the ground-water level in the sandy strata being lowered by continuous pumping from a series of artesian pipes driven so as to encircle the excavated area. After the concrete work was finished, the water was again permitted to enter the lock-pit and the back-filling was placed by suction dredges in the manner commonly used in hydraulic-fill dam construction.

The masonry structure had to be sufficiently strong to withstand the greatest pressure at all times, during the placing of the back-filling and after it was entirely completed. The correct determination of these pressures, at all stages of the work, therefore, was very important. In the beginning, there was a purely hydrostatic pressure equal in the horizontal and vertical directions. This condition changed gradually as solid matter was added, the vertical component increasing more rapidly than the lateral component. Subsequently, there was further consolidation by drainage and the drying of the material which gradually lost its fluid or semi-fluid properties, so that finally the horizontal pressures were very much less than the vertical pressures.

The experiments at New Orleans were undertaken in order to ascertain, as far as could be done with a model, the intensity of lateral pressure produced by soft material deposited under conditions similar to those in the actual construction of the lock. Scant information of this kind was available at the time, although the core studies on the hydraulic-fill dams of the Miami Conservancy District have since been published, which furnish valuable information regarding the condition and behavior of such material.†

The studies at New Orleans were made by R. O. Comer, Designing Engineer of the Port Commission, and embodied in a "Report on the Physical Characteristics of New Orleans Blue Clay with Reference to the Backfilling Operations in the Lock", from which the following information has been abstracted.

The apparatus used is shown in Fig. 23, and consists of a strongly framed box 2 ft. square in horizontal section and 6 ft. high, carefully built of matched lumber and made absolutely water-tight on three sides by a heavy coating of roofing pitch.

The front of the box was made in three parts—two $\frac{7}{8}$-in. yellow pine heartwood pieces, 8 in. wide, and a steel plate, 8 in. by $\frac{1}{2}$ in.—all 6 ft. long. The

* "The Lock Foundation, Inner Harbor Navigation Canal, New Orleans: The Problem and the Solution", by George R. Goethals, M. Am. Soc. C. E., *Proceedings*, Louisiana Eng. Soc., Vol. VI (1920), p. 134.

† "Core Studies in the Hydraulic-Fill Dams of the Miami Conservancy District," by Charles H. Paul, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1181.

entire front is movable, the two boards and the plate being free to move with reference to each other and the sides of the box, but prevented from outward displacement, at the top and bottom, by the framework against which they rest. To prevent leakage, the entire inner surface of the two boards and the plate was covered with canvas which was left loose at the corners to allow freer movement and prevent "arch action" in the clay as far as possible.

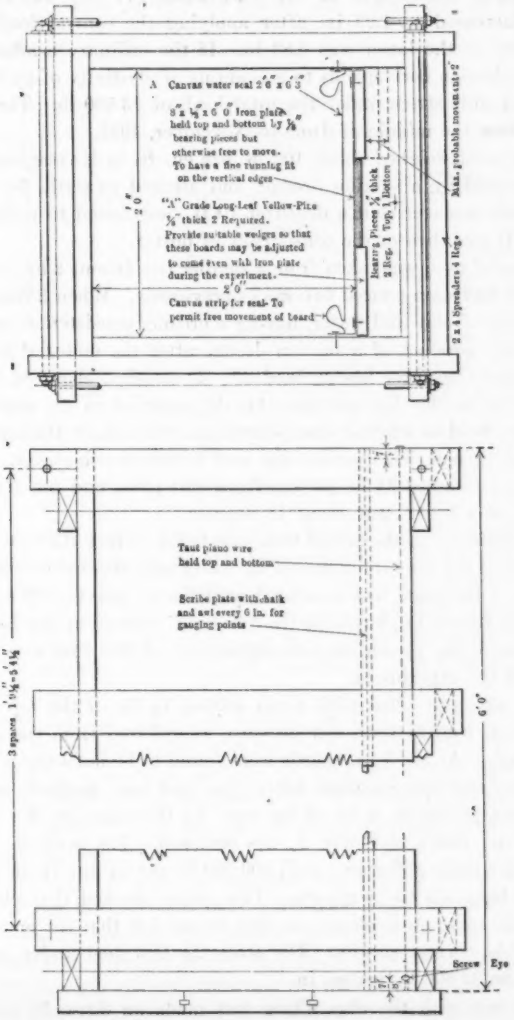


FIG. 23.

The lateral pressures under different conditions of loading were derived from the measured deflections of the steel plate which was calibrated before being attached to the box. The calibration was done by placing the plate horizontally on two supports 6 ft. apart, and subjecting it to a transverse load of 219 lb. at the middle of the span, the deflections being measured from a piano wire stretched over the center line of the plate. The observed maximum deflection under the weight of the plate alone (77 lb.) was 0.42 in. This deflection increased to 1.25 in. after applying the central load, so that the deflection due to the latter was 0.83 in. If the ordinary bending formula is used, this deflection corresponds to a modulus of elasticity of 28 700 000 lb. per sq. in. and a unit stress under the total load, of 15 500 lb. The experiments continued from the middle of June to November, 1921.

The clay used weighed about 100 lb. per cu. ft. in its original bed. After it was excavated by a suction dredge, and allowed to settle for two years, a sample freshly uncovered at a depth of 10 ft., was found to weigh only 93 lb. per cu. ft. It was then of the consistency of butter.

The material used was taken from a smaller box (about 2 by 5 by 5 ft. high) into which it had been poured two weeks previously. When filling this smaller box, a mixture of clay and water, having a creamy consistency was used, so as to simulate the product of a suction dredge after the material has settled for a few days and the water has drained off. It weighed about 85 lb. per cu. ft. When taken from this box on June 14, the material in the central part was apparently as fluid as when it was poured in, although on the top, the bottom, and the sides, it had dried perceptibly and become much stiffer. A bucketful of the mixture weighed 91 lb. per cu. ft. at this time, and the angle of repose, when tested on a board, was about 18 degrees.

As a preliminary to the actual tests and before filling the box, a piano wire was stretched along the center line of the plate, and the distance from the wire to the back of the plate was measured at different points. The shape of the plate, as thus found, is plotted in the "straight" curves at the bottom of Fig. 24, two curves being shown, as a re-adjustment of the wire was made during the course of the experiment.

The box was next filled with water within 1½ in. of the top and the full curve shown on Fig. 24 for water pressure was plotted from the deflections at different points. As the ⅞-in. boards were found to be not exactly in line with the plate, this test was repeated, after they had been properly adjusted, and the water brought within ⅞ in. of the top. In this manner, the center deflection of 1.83 in., shown in Curve *A*, was obtained. The modulus of elasticity, corresponding to this deflection, is 27 500 000 lb. per sq. in., if the plate is considered as a beam on two supports. This value checked that obtained when calibrating the plate quite closely, so that it was felt that the apparatus would give reasonably correct results. The stress in the steel under this load was computed to be 22 800 lb. per sq. in.

The first test with the clay filling was made on June 15 and numerous other tests were made during the next four months. After the larger box had

been filled on June 14, it was found that 5 lb. sufficed to force a wooden stick 1 in. square in section, rapidly to the bottom through 6 ft. of material.

On June 15, readings were taken on the steel plate in the same manner as that used when the box was filled with water. The results are shown in Curve *B* in Fig. 24. As the maximum deflection was 2.1 in., whereas for water it was 1.83 in., the weight of a fluid producing an equivalent lateral pressure was found to be $\frac{2.1}{1.83} \times 62.4$, or 71.6 lb. per cu. ft. The stress in the plate was about 26 200 lb. per sq. in.

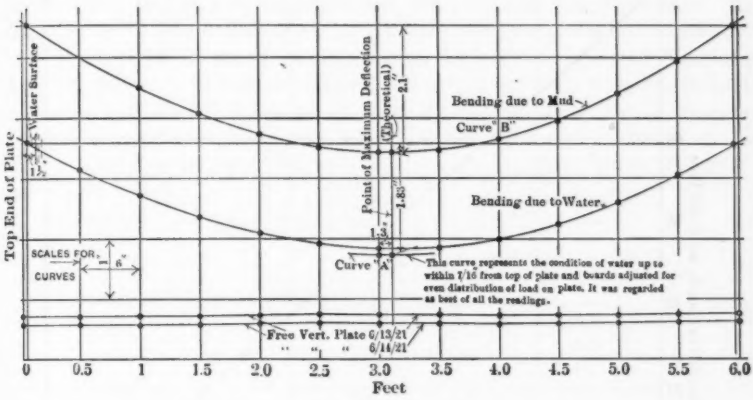


FIG. 24.

The results of this test are believed to be quite accurate, as the material in the box was thoroughly stirred with a rod before the readings were taken, so that it is thought that no stresses could exist in the body of the material itself sufficient to introduce serious inaccuracies. Later, when the material became stiffer, this conclusion was not so fully warranted, as the material could not be easily rodded. However, the maximum pressures which occurred during the earlier tests, were of greater practical importance.

After each test, the plate was pushed back as nearly as possible to its original position by small jacks attached to the framework, and especial care was taken to have the inner surfaces of the boards and plate in the same plane, at the beginning of each experiment. This was found to be of extreme importance, to prevent "arching" of the material, especially when it became stiffer through lapse of time.

After several weeks, when the material had become stiffer, it was noticed that three or four days elapsed before the plate took its maximum deflection for any particular reading. In other words, the pressure exerted by the material seemed to increase for several days after the jacks were released. The results of the several measurements are shown in Fig. 25, which gives the "equivalent fluid weights", that is, the weights of fluids which would produce the same lateral pressures as the material tested.

When first placed in the box, the material was a fluid that could readily be poured from a pail and weighed 85 lb. per cu. ft. As time went on, it dried out and consolidated. The box stood in the open air from June until November, with a piece of canvas over the top. The filling settled perhaps as much as 18 in. As the material settled, fresh material was added from time to time from the smaller box, so that when the readings were taken, the full head was always acting.

The reduction in the equivalent fluid weights from 73 to 25 lb. per cu. ft.

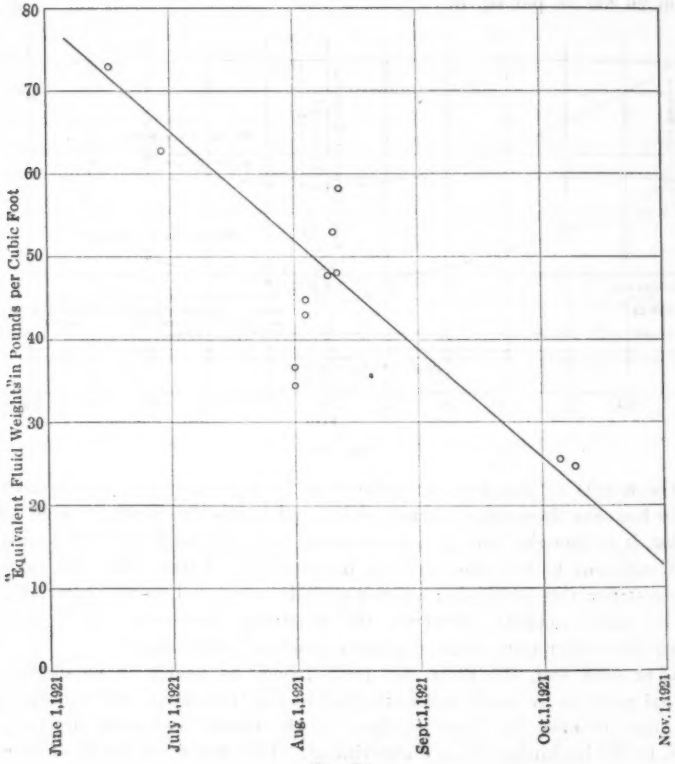


FIG. 25.

as time progressed, shows clearly the gradual stiffening of the clay. The points fall somewhat irregularly, especially about August 1, when the clay had stiffened considerably. Those closest to the inclined line joining the maximum and minimum pressures are believed to be the most reliable. Although measurements made with the somewhat crude apparatus used, cannot be expected to give very accurate results, the experiments were of much assistance in judging the effect of the back-filling. As the result of the experiments, it is believed that, for material of this kind, the active lateral earth pressure will

never exceed that due to a fluid weighing 75 lb. per cu. ft. When the material is more nearly fluid, it weighs less, and when it is stiffer, it exerts relatively less lateral pressure.

The lower 49 ft. of the material behind the lock walls was clay deposited by a suction dredge, and the 16 ft. at the top was sand, part of which was placed by pumping and part as a dry fill. The total height of 65 ft. extends from the top of the wall to the bottom of the floor-slab. The pressures, as shown graphically in Fig. 26, are believed to be an entirely safe guide in designing. The sand and clay were both assumed to weigh 100 lb. per cu. ft. and to be equivalent in their effect to fluids weighing, respectively, 20 and 55 lb. per cu. ft. The clay is supposed to have become somewhat dried out, when the sand was placed on it. At Elevation + 29 (the bottom of the sand), the latter produces a lateral pressure of 20 by 16, or 320 lb. per sq. ft. In the clay, the superimposed sand (weighing 1 600 lb. per sq. ft.) produces a lateral pressure of 880 lb. per sq. ft., and the maximum pressure is 3 575 lb. per sq. ft. at the bottom. This value is the same as would be produced if the clay, which has the same specific weight as the sand, extended to the top of the wall, although the local pressure in the uppermost 16 ft. is less.

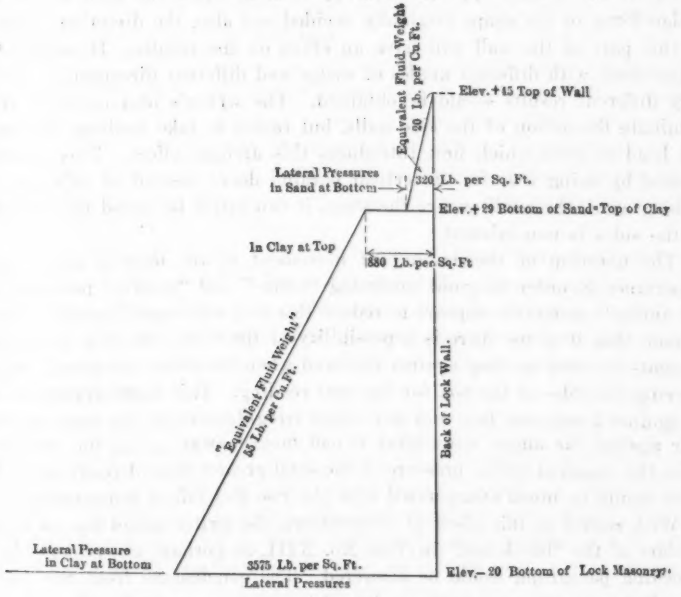


FIG. 26.

In reality, there was no sharp plane of demarcation between the sand and clay. Near the wall the sand sank 6 or 8 ft. into the softer material below, whereas at some distance from the wall the clay rose above its original level

producing what was known as a "mud wave". The pressure against the wall, therefore, would change gradually, without the sharp jog shown in Fig. 26.

T. E. N. FARGHER,* ESQ. (by letter).—The writer at present is engaged on experiments somewhat similar to those described by the author, but artificially dried sand is being used. There are several points of particular interest regarding the experiments. The author states that "the coefficient of friction is determined by measuring the force required to move a known mass of material resting on a plane of the same material". Two coefficients were obtained, one to start the motion, and one to keep the mass moving uniformly. The writer would like to see a more detailed description of the apparatus used, as he tried similar experiments on dry sand, and found that it was impossible to obtain uniform motion, because as soon as motion took place to any extent the box tipped down in front and began to plow up the sand, thus introducing another factor which, of necessity, ended the experiment. As there is probably little cohesion in dry sand, this difficulty would not affect this material.

The idea of a rolling side-wall to overcome the arching effect of friction on the sides is ingenious, but appears to be rather like making the experiments fit the theory. It will naturally tend to cause the sand to slip in a wedge form, of the shape originally decided on; also, the direction of motion of this part of the wall will have an effect on the results. It seems to the writer that, with different angles of wedge and different directions of motion, very different results would be obtained. The writer's idea is not to try to eliminate the action of the side-walls, but rather to take readings only up to the head of sand which first introduces this arching effect. This is accomplished by using a series of vertical parallel doors instead of only one, and as long as all the readings are the same, it can safely be stated that the effect of the sides is non-existent.

The question of the amount of movement of the door is one of great importance in order to avoid confusing "active" and "passive" pressures, and the author's apparatus appears to reduce this to a very small amount, so small in fact, that it seems there is a possibility of the 6 by 4-in. stop angle which prevents the door moving against the sand when the riders are moved out, not clearing the sides of the box for the next reading. This angle appears to bear up against a concrete face and any slight irregularities in the concrete would bear against the angle, even after it had moved away $\frac{1}{16\,000}$ in., and would make the apparent active pressure of the sand greater than it really was. This effect would be much exaggerated with the rise and fall of temperature.

With regard to this effect of temperature, the writer would like to see particulars of the "blank test" in Test No. XIII, as perhaps the remarks in the preceding paragraph would be answered. The conclusions from this test, as regards temperature, might be explained by the increase in length of the connecting links with a rise of temperature, which would tend to allow a lesser movement of the door and would magnify any effect of the "stop angle" not clearing the sides of the box. This effect would be greater at the top horizontal

* Liverpool, England.

support than at the bottom one and thus would cause the point of application of the resultant to rise with a rise of temperature and fall with a fall of temperature.

The writer would like to know if an attempt was made to determine the changes in temperature in the interior of the sand, as it appears that the changes of outside temperature would have far more effect on the apparatus than on the interior of the sand, where it would have to be to cause any difference in the actual active pressure of the sand. The writer's experiments are not sufficiently advanced to make any definite statement regarding his conclusions, but the increase of apparent pressure with the temperature in his experiments, is quite marked, but so far can be explained by the effect on the apparatus alone.

Other points of interest, but more for the information of future experimenters, are the questions of the actual filling of the box, and how any particular level was obtained, as any shoveling on the surface would cause small irregularities, and dumping the material in from a height of 6 to 8 ft. would have an appreciable effect on the readings of even such a large apparatus as that described. The dry sand that the writer is using, can be increased 10% in density by shaking or ramming, and damp sand also behaves similarly, although perhaps not to such an extent.

It would be of interest if the author would give some idea of the reasons for the non-uniformity of the higher readings on Scales Nos. 2 and 3 in Test No. 1 (Columns 8 and 9 of Table 3), and whether these give an idea of the type of readings obtained with the other experiments. The readings on the vertical Scales Nos. 4 and 5 are in excellent agreement. The writer has found with his apparatus that the vertical readings are not, in general, in such close agreement as the horizontal ones.

The very able way in which the author has conducted his experiments and collected the results of others has made this paper of outstanding interest, and the manner in which many previously debatable subjects, such as the direction of the resultant force, effect of concentrated surcharged loads, age of fill, etc., have been dealt with, has added considerably to the knowledge of this important subject.

P. M. CROSTHWAITE,* ESQ. (by letter).—The writer has read with great interest the investigation described by Mr. Feld, and must express his admiration for the experimental ingenuity displayed in the design of the apparatus.

Toward the end of the paper, Mr. Feld deals at some length with the writer's experiments.† From these experiments, the conclusion was drawn that friction between the backing and wall did not affect the resultant thrust. Mr. Feld disputes this conclusion on the ground that a rough wall was used, and states "Had the experimenter used a smooth wall and obtained the same results, the conclusions might have an experimental foundation."

* Highfield, King's Langley, Herts, England.

† "The Horizontal Pressure of Sand", *Minutes of Proceedings*, Inst. C. E., Vol. CCIX, Pt. 1.

The reason for using a rough wall was that the writer wished to obtain friction, and the rougher the wall the greater the friction. Had the wall been perfectly smooth, there could have been no friction between it and the backing, hence, the writer still thinks he was perfectly justified in using a rough wall.

Mr. Feld states that in some of the writer's experiments the results are widely scattered, many readings being 50% from the mean. This conclusion does not appear to be in accordance with the facts. For example, the mean of the last nineteen experiments of Series 1, all of which are for wedges that give a maximum pressure against a wall backed up by loose dry sand, is 0.1916 for the ratio of the horizontal to the vertical pressure. The maximum result for any one experiment was 0.213, and the minimum 0.181; that is, the experiments varied from 5.7% above to 6.1% below the mean, and all the other experiments show equally consistent results.

Mr. Feld differs with the writer for expressing opposite conclusions in his papers of 1916 and 1920, and states that had he taken the trouble to measure the angle of internal friction of the material he was using he would have found it always less than the angle of repose. These experiments were conducted in order to measure the angle of friction and nothing else, and it was found that three distinct values were obtained, according to how the measurement was made. One result was obtained by the plunger method, a second by measuring the pressure against a wall, and a third when it was measured by shearing. The first two values were considerably greater than the angle of repose, and the third value was considerably less. This is fully explained in the 1920 paper, and must have been inadvertently overlooked.

Mr. Feld states that he cannot understand why the writer doubted the truth of the wedge theory, which takes in wall friction, seeing that the experiments and calculated results agree. As a matter of fact, they did not agree. It is true that the wedge theory with wall friction gave correct results for a wall without surcharge, but when applied to a surcharged wall, the results were incorrect when the angle of repose was used for calculating the thrust.

It is quite evident that no theory can be correct unless it applies both to a surcharged wall and a wall without surcharge, and the writer found that by using an angle of internal friction, as determined by measurements of the pressure against a wall without surcharge, he could calculate the correct thrust produced by a surcharge, provided he neglected wall friction. For this reason he adopted the modified theory, which neglects wall friction.

With regard to the author's experiments and conclusions, it is true that the forces applied to the experimental wall by the thrust produced by the fill must be balanced by the external forces acting on the wall, and it is evident that the horizontal component of the thrust is accounted for by the pressure on Gauges Nos. 1, 2, and 3.

The author, however, assumes that the vertical component of the thrust is balanced by Gauges Nos. 4 and 5. It does not appear to the writer that this assumption is justified, because another vertical force acts on the wall, namely, the friction between it and the fill, as the wall must have moved down-

ward past the fill before Gauges Nos. 4 and 5 could record any pressure, and this friction must have been overcome. It is very difficult, therefore, to understand why Gauges Nos. 4 and 5 should have recorded any pressure until the depth of fill was considerable, whereas they began to record pressure when the depth of fill was only 5 in.

Take, for example, Test No. 1 (Table 3): With a 72-in. depth of fill, the total horizontal pressure is 2 273 lb. Assuming the angle of wall friction to be 30°, the total friction between the wall and the fill would be $2\,273 \times \tan 30° = 1\,311.5$ lb.

The total vertical forces, acting on the wall, that balance the vertical component of the thrust, therefore, would appear not to be 1 495.5 lb., as given in Column 16, Table 3, but $1\,495.5 + 1\,311.5 = 2\,807$ lb. If this reasoning is correct, the total resultant thrust on the wall would be $\sqrt{2\,273^2 + 2\,807^2} = 3\,612$ lb., and not 2 721 lb. as given in Column 17, and the ratio of $\frac{V}{H}$ would be $\frac{3\,612}{2\,273} = 1.59$, not 0.659 as given in Column 21.

It should be noted, further, that if this friction is taken into account, it is impossible to calculate ϕ' from the experiments, because in order to ascertain the total friction, it is necessary to assume a value for ϕ', taken as 30° in the calculations just mentioned.

Darwin, in his paper on the "Horizontal Thrust of a Mass of Sand",* mentions the difficulties caused by the straining and deformation of the door. It is evident that bending moments and distortion must be produced in any door or wall used in such experiments, and it is possible that gauges beneath the door might be affected by such deformations, apart from any vertical force due to the thrust of the sand.

From the description of the apparatus, it appears that the movement of the platform was reduced to $\frac{1}{1000}$ in., so that an extremely small deformation of the wall might affect Gauges Nos. 4 and 5 without the existence of any vertical components.

In Appendix II of the writer's paper of 1920, he showed that, if the resultant pressure from a surcharged wall is parallel to the surface, as assumed by Rankine, it should not produce any pressure on the foundations under the heel of the wall, because it is balanced by friction. If this is true, it should likewise not have thrown any pressure on Gauges Nos. 4 and 5, as they are merely substitutions for the foundations.

From the description of the author's experiments, it will be noticed that between each additional fill "the riders on the scale-beams were moved out, that is, the scales were overbalanced, to a point more than the next reading." This must have produced a bending moment in the wall, the amount of which would depend on the distance the riders were moved. This bending moment must have produced some distortion of the door, and, as the riders were moved forward again, after the next layer of sand had been put in, and the forces that had bent the door were relaxed, the pressure indicated on

* *Minutes of Proceedings*, Inst. C. E., Vol. LXXI, Pt. 1.

Gauges Nos. 2, 3, and 5 would be in excess of that due to the static pressure of the sand.

It may be said that because the door was very rigidly braced the distortion due to the bending moment was extremely small. However small it might have been, it must have produced the result just indicated, which would have tended to make readings on Gauges Nos. 1, 2, and 3 too high. This is illustrated in Fig 27, on which the arrows at the left side represent the excess pressure thrown on the wall by moving out the riders on the scale-beams, causing the door to apply an active pressure on the sand. In fact, the condition of the wall in these circumstances is somewhat analogous to that of a bent bow, which tends to straighten itself when the tension at its ends is released.

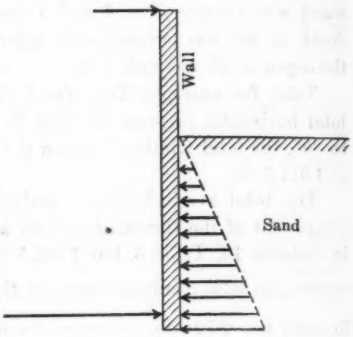


FIG. 27.

It is satisfactory to the writer to see that in the general conclusions given on page 1493, it would appear that the results obtained from the Cincinnati experiments confirm the conclusions he had drawn from his own experiments. That is (*a*), no defined plane of rupture was found; (*b*) the resultant pressure acts from one-third to four-tenths of the depth of the fill, as compared with one-third obtained by the writer; (*c*) that for walls without surcharge, the Rankine or Coulomb theories hold true, provided ϕ is the angle of internal friction and not the angle of repose; and (*d*) that for vertical walls, wall friction should be neglected.

On page 1501, Mr. Feld refers to the similarity of the conclusions in the writer's paper and those he has drawn from his investigation, but doubts the writer's right to draw these conclusions from his experiments. The only remark the writer can make on this criticism is that if he were re-writing the paper, he could draw no other conclusions.

The main difference between the conclusions of Mr. Feld and those of the writer is that, in the Cincinnati experiments, the coefficient of internal friction is measured by shearing, whereas in the writer's experiments, it was deduced by measuring the pressure against a wall without surcharge. It is still thought that this is the more rational method, as this is the way the material is actually being used.

In order to determine the tensile strength of steel, one would not test it for compression or shearing, and there seems to be no reason for treating other materials differently, especially when it is known that for sand there are three perfectly distinct angles of internal friction, depending on how it is measured. For loose sand, they are:

By plunger method	51½°
" wall "	42° 20′
" shearing "	23°

corresponding to coefficients of friction of 1.257, 0.911, and 0.424. For well stirred sand, the corresponding coefficients are 1.880, 1.150, and 0.753.

Darwin's experiments of 1877, referred to by the author were on a small scale, but it is hardly fair to state that they are "unreliable." They were repeated by the writer with almost identical apparatus, and were confirmed in nearly every particular, practically identical results being obtained for surcharged walls, walls without surcharge, and walls with negative surcharge. These results were checked by the experiments with the rolling wall, an entirely different method, and the agreement between both sets of experiments was complete.

On page 1499, Mr. Feld quotes the writer, as follows: "The angles of internal friction from these experiments was much less than the angle of repose. * * *". This is exactly the opposite of what was actually in the paper, in which it was stated: "In both the present [*i. e.*, wall experiments] and in the author's plunger experiments he found the angle of internal friction of dry sand to be considerably greater than the angle of repose."

It was only when the angle was determined by shearing that it was less, and this is referred to particularly and at considerable length in the writer's reply to the correspondence because, until the experiments were made, it had never occurred to him that there might be three quite distinct coefficients of internal friction.

JACOB FELD,* JUN. AM. SOC. C. E. (by letter).—The writer is grateful to those who have taken part in the discussion. The constructive tone of the comments is most pleasing. He is especially interested in the remarks relative to a soil physics and a physical classification of soils, as he is compiling all the existing experimental data on the characteristics of materials called by such names as granular, pulverulent, semi-liquid, semi-solid, cohesive, plastic, etc., which do not fall readily into any one of the following classes: Solid, liquid, or gaseous. Numerous attempts have been made to classify such materials, starting with Querlonde's† division of soils, in 1743, into vegetation soil, stiff earth, and sand, the criterion being the natural slope.

C. C. Williams, M. Am. Soc. C. E., has recently classified‡ the non-solid and non-liquid materials as follows:

1.—Granular materials, in which pressure increases with the depth; nowhere can there be an upward component, and at all points the ratio of vertical to horizontal pressure is a function of the angle of internal friction.

2.—Fragmentary materials, in which, after a certain depth, there is no lateral pressure, for the material acts as a solid after the rough particles are held together by intercogging under load.

3.—Plastic materials, in which the ratio of vertical to horizontal pressure is a constant at all depths.

4.—Viscous materials, almost liquid.

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* Brooklyn, N. Y.

† Mayniel, "Traité Expérimental, Analytique et Pratique de la Poussee des Terres et des Murs de Revêtement", 1808.

‡ "Design of Masonry Structures", 1922, pp. 249–274.

Mr. Meem considers five classes as sufficient. The writer expects the next classification to consist of six groups and, mathematically speaking, the number of individual types being infinite, the number of classes approaches infinity as a limit, as the classification becomes more accurate. A perfect classification, by the method of grouping like materials, is impossible, because the list of individual materials comprises a series of discontinuous terms whereas the properties vary according to a continuous function. For example, clay denotes a material having a high degree of cohesion; it falls into the class of plastic materials in the Williams' system, and into the cohesive or semi-cohesive class, according to Mr. Meem. As there are clays of varying cohesive properties and clays with almost no cohesion, the classification is not general. Similar results would be obtained by attempting to classify almost any other state of matter by this method. An analogous case occurs in the classification of solids, if each property is denoted by a ratio to a standard unit (elasticity, density, etc.), instead of grouping the materials into classes, as hard, soft, brittle, etc.

The stresses acting in a granular mass in a state of equilibrium are caused by the forces of gravity and gravitation (attraction between the earth and the particles and between the particles themselves), surface tension, adhesion, molecular attraction, etc. These stresses are usually grouped into internal friction and cohesion, which, together with density, the measure of gravity, form the basis of discrimination between materials. However, such a simple classification, although desirable, is not feasible, because even if it has been proved that the friction and cohesion are independent of each other,* they are not determined so easily and they do not include all the possible differences between materials. For example, a clay and a loamy sand containing some vegetable matter may have the same unit weight, internal friction, and cohesion, but they are not the same material, and will not act in the same manner behind a retaining wall or beneath a footing. After these properties are determined, there is no certainty that a slight change in conditions may not cause a very great change in the property. For example, there are many materials in which friction acts in accordance with the laws of friction for solids until a small quantity of water causes a lubricating action, and then, for a true analysis, the laws of friction for lubricants must be used. Professor E. C. Bingham† tabulates the laws governing the friction of solids and lubricants to show the differences between them. He states that the laws of friction for solids are as follows:

1.—The frictional resistance is equal to a constant multiplied by the normal force acting; the constant is the coefficient of friction.

2.—The static friction, at incipient motion, is always greater than the dynamic friction, during motion. (In the paper, these quantities are termed "internal resistance" and "internal friction", respectively.)

3.—The frictional resistance is independent of the area in contact.

* Leygue, "Nouvelles Recherches sur la Poussee des Terres et le Profil de Revêtement de Plus Economique", 1885, *Annales des Ponts et Chaussées*, Vol. 10, pp. 780–1000; Resal, "Poussee des Terres et Stabilité de Revêtements", 2 vol., Encyclopédie des Travaux Publics, 1903, '10.

† "Fluidity and Plasticity", 1922.

He gives the following laws of friction for viscous flow:

1.—The frictional resistance is independent of the load.

2.—The frictional resistance is directly proportional to the velocity, and vanishes when the velocity is zero.

3.—The frictional resistance is directly proportional to the area in contact.

No better example could be desired by which to illustrate the absolute uncertainty that results from using the friction as a criterion for classification. The Special Committee on the Bearing Value of Soils for Foundations, etc., has recently announced its belief that the colloid content is an important factor in the action of soils. In a lecture before the Chemistry Seminar of the University of Cincinnati, in 1920, the writer showed the importance of the colloid matter in soils, and demonstrated how all the properties of the soil in the vicinity—a decomposition product of the Cincinnati limestone-shale formation—could be explained by the colloidal properties of the constituents.

The writer believes that the classification of soils and other granular materials should be along the lines of strain characteristics rather than those of stress, namely, elastic strain, plasticity, and fluidity, for these phenomena are evident and measurable, and embrace within them the results of all the stresses which may act. Generally speaking, the elasticity factor will take care of the solid, the fluidity of the fluid, and the plasticity of the colloid properties of the material being classified.

Elasticity is best described by Hooke's law, *Ut tensio sic vis*, the axial deformation is directly proportional to the axial stress, and Poisson added that this deformation is accompanied by two strains of opposite kind along the other axes. It is not necessary to go into the details of elasticity in this discussion.

The properties of fluidity and plasticity have been discussed thoroughly by Bingham,* with, however, too little emphasis on these properties in solids and granular materials. His description is summarized in the following definitions:

If a perfectly elastic body is subjected to a shear, a strain develops, which disappears entirely when the stress is removed. The total work done is zero; there was no change in volume, although there was a change in shape. This is a case of elastic deformation. In imperfectly elastic bodies, part of the deformation will remain, and some work has been performed in overcoming internal friction. Such a deformation or flow may be of three kinds:

1.—Viscous or linear flow, in which the deformation is proportional to the deforming force. A certain time interval may be required to show the constancy of the relation.

2.—Hydraulic or turbulent flow, usually in fluid materials.

3.—Plastic flow, in which the shearing force starts a deformation which continues indefinitely as long as the force remains.

The mathematics of this subject will not be introduced at present.

* "Fluidity and Plasticity", 1922.

The writer believes that a scientific and adequate classification of materials not easily classed as solid, liquid, or gaseous, can be made by assigning numerical values—ratios to fixed arbitrary units—for these properties. In this manner, a gradual continuous variation along three axes may be denoted. It is possible that there are more than three independent variables which are sufficiently important to be considered, but the writer does not believe that any one of these three can be eliminated.

An investigation of these properties will show that they depend on the various characteristics of the material, among which are included the specific gravity, the percentage of voids, the moisture content, the mechanical analysis (including the percentage content of particles of colloid size), the chemical constituents, the shape of the particles, the percentage of organic matter, etc. Not until the laws of variation of the three factors have been fully determined can a complete and adequate soil mechanics be developed. Such an undertaking is too much for an individual, and it is hoped that the proper syllabus will be drawn up, through the agency of the Society, to divide and co-ordinate research along this line.

The writer's method for determining the "coefficient of internal resistance" is a simple, yet fairly accurate, means for measuring the resistance to internal shear, a measure of the fluidity of the mass. The work required is more of an extensive nature than the usual intensive researches in physics; too great a refinement in this work is useless, especially as it must not be conducted with small-scale apparatus.

Numerous theories have been developed for the evaluation of the lateral pressure of granular materials, but each assumes that the material is a solid. Especially is this true of Rankine's theory, which assumes that the material is a homogeneous elastic mass up to the value of the internal friction, beyond which, it becomes a perfect liquid.

The "rational" elastic theory of Levy, St. Venant, and Boussinesq, is merely a refinement of the Rankine theory, the addition of factors covering the end conditions at the wall and at the surface of the fill.

The wedge theory assumes a solid wedge of earth, acting as a unit, an assumption which is equivalent to that of the elasticians. That this is true can be seen from the fact that the general wedge formula can be converted into the Rankine formula by algebraic transformations, if it is assumed that the direction of the resultant is always parallel to the top surface.*

The only difference between the two theories, therefore, is the assumption of the resultant direction. Experiments have proved which assumption is correct in the case of a retaining wall. The theory that will give the magnitude and point of application correctly has not yet been developed. The wedge theory is a close approximation for materials which have but little fluidity and plasticity, and in which the elastic properties are the prevailing characteristics. The theory that will include all materials will include all their variable factors.

* Cain, "Practical Designing of Retaining Walls", 1888 Edition, or Paaswell, "Retaining Walls", 1921.

For a comparison of the theories, the coefficients of $\frac{1}{2}yh^2$ are given in Table 40 for the following cases: Angle of internal resistance, 40°; angle of wall friction, 30°; values of α, — 10°, 0 (vertical wall), and + 10°; values of ϵ — 30°, 0 (level fill), and + 30°. The theories used were:

1.—The Français theory (1820): The pressure exerted by a wedge is always normal to the wall. The original Coulomb formula is a special case of the Français formula.
2.—The Poncelet theory (1840): The pressure is always inclined to the wall normal by the angle of wall friction.
3.—The Rankine theory (1856): This applies to certain cases only.
4.—The Boussinesq theory (1883): The accurate solution of Levy's "rational" theory.
5.—The Flammant formula (1884): An approximate simplification of Boussinesq's result.
6.—The Chaudy stress transmission theory (1895).
7.—The Hummel and Finnan empirical solution (1922): Based on experimental data.

The values given are E, H, and V, the coefficients of $\frac{1}{2}yh^2$ for the total, the horizontal, and the vertical components, respectively, and δ, the angle which the resultant makes with the horizontal.

An inspection of Table 40 shows that the theories differ only slightly in evaluating the total or the horizontal pressure, the main difference being in the vertical component.

In reference to Mr. Meem's arching theory, the writer believes that, basically, it is the same as the wedge theory. The differences in the results are due to differences in the assumptions of the direction and height of the resultant. His reason for this belief is the fact that the arch, considered as a static unit, is merely a solid wedge, one of the fundamental machines. If the arches in Mr. Meem's theory are summated, from the bottom to the top of the wall, the result is a solid wedge, the assumption of the Coulomb theory. The difference is really in the assumption of the direction of motion of the grains as failure occurs; the wedge theory assumes that the particles tend to move parallel to the plane of rupture, and, therefore, the resultant motion acts through the center of gravity and parallel to the plane of rupture, intersecting the wall at $\frac{1}{3}h$ above the base; the arching theory assumes that the particles tend to move horizontally (a flattening of the arch), and, therefore, the resultant motion acts horizontally and intersects the wall at $\frac{2}{3}h$ from the base.

The experimental work of Leygue,* who watched the motion of the sand grains in a glass box as one side was tipped or displaced by translation, and the photographic investigations by Mueller-Breslau† of the same phenomenon on a larger scale, have shown conclusively that no matter what the nature of the failure of the wall, the particles move in lines parallel to the line of

* *Annales des Ponts et Chaussées*, 1885.

† "Erddruck auf Stützmauern", 1906. A translation of this experimental work will be found in the writer's thesis, a copy of which is filed in the Engineering Societies Library.

rupture. This is true for all points in the wedge. In view of this, the writer is inclined toward the idea of a wedge of rupture, rather than the arching idea, for the case of active lateral earth pressure.

TABLE 40.

		$\alpha = -10°$.			$\alpha = 0°$.			$\alpha = +10°$.		
	ϵ.	−30°	0.0	+30°	−30°	0.0	+30°	−30°	0.0	+30°
Français*	E	0.132	0.159	0.224	0.174	0.219	0.318	0.218	0.290	0.432
	H	0.130	0.157	0.221	0.174	0.219	0.318	0.215	0.285	0.425
	V	−0.022	−0.025	−0.038	0	0	0	0.037	0.050	0.074
	δ	−10°	−10°	−10°	0	0	0	10°	10°	10°
Poncelet†	E	0.113	0.139	0.209	0.156	0.210	0.316	0.204	0.282	0.463
	H	0.106	0.130	0.196	0.135	0.181	0.274	0.156	0.215	0.355
	V	0.041	0.051	0.076	0.090	0.121	0.188	0.131	0.181	0.298
	δ	20°	20°	20°	30°	30°	30°	40°	40°	40°
Rankine‡	E	.....	.....	.....	.....	0.219	0.288	.....	.....	.....
	H	.....	.....	.....	.....	0.219	0.250	.....	.....	.....
	V	.....	.....	.....	.....	0	0.144	.....	.....	.....
	δ	.....	.....	.....	.....	0	30°	.....	.....	.....
Boussinesq§	E	0.045	0.123	0.212	0.130	0.205	0.336	0.208	0.300	0.511
	H	0.042	0.116	0.200	0.113	0.178	0.291	0.156	0.230	0.372
	V	0.015	0.042	0.073	0.065	0.103	0.168	0.131	0.198	0.329
	δ	20°	20°	20°	30°	30°	30°	40°	40°	40°
Flammant‖	E	.....	.....	.....	.....	0.228	0.353	.....	0.309	0.513
	H	.....	.....	.....	.....	0.174	0.270	.....	0.199	0.330
	V	.....	.....	.....	.....	0.146	0.227	.....	0.235	0.395
	δ	.....	.....	.....	.....	40°	40°	.....	50°	50°
Chaudy¶	E	.....	0.18	.....	.....	0.26	.....	.....	0.35	.....
	H	.....	0.095	.....	.....	0.12	.....	.....	0.135	.....
	V	.....	0.153	.....	.....	0.23	.....	.....	0.322	.....
	δ	.....	58°15′	.....	.....	62°30′	.....	.....	67°15′	.....
Hummel and Finnan**	E	.....	.....	.....	0.172	0.218	0.297	.....	.....	.....
	H	.....	.....	.....	0.172	0.218	0.297	.....	.....	.....
	V	.....	.....	.....	0	0	0	.....	.....	.....
	δ	.....	.....	.....	0	0	0	.....	.....	.....

* Français, "Recherches sur la Poussee des Terres", 1820, *Mémorial* de l'Officier du Génie, pp. 156–251.

† Poncelet, "Mémoire sur la Stabilité des Revêtements", 1845. For the theory, see any standard work on earth pressures.

‡ Rankine, "Applied Mechanics", 1858; or, "On the Stability of Loose Earth", 1856, *Philosophical Transactions*, Royal Soc., Lond., pp. 2–27.

§ Boussinesq, "Essai Théorique sur l'Equilibre des Massifs Pulverulents", 1876, Brussels, Mm. Cour, Vol. 40, No. 4. Also, *Comptes Rendus*, Academy of Sciences, 1884, Vol. 98, pp. 720–723, 790–793, 667–670, 901–904, 975–978.

‖ Flammant, "Tables Numériques pour le Calcul de la Poussee des Terres", 1885. *Annales des Ponts et Chaussées*, Vol. 9, pp. 515–540.

¶ Chaudy, "Résistance des Terrains Sablonneaux", 1895, *Mémoire et Compte Rendu* des Ingénieurs Civils, pp. 607–617.

** Hummel and Finnan, "Distribution of Pressure on Surfaces Supporting a Mass of Sand", 1921, *Minutes of Proceedings*, Inst. C. E., Lond., Vol. CCXII, pp. 369–392.

For the case of timbering in cuts, where the wall is braced horizontally against the fill, a different condition may exist in the mass, but no experimental investigation of this case, the passive resistance to pressure, has been made. The movement in this case, as compared with that of active pressure, will have to be quite large, in order to take up the "slack" or excess voids in the material. A measure of this slack is the ratio of the percentage of voids, as ordinarily determined, and the percentage of absolute voids, found

when the particles are touching, that is, when the mass occupies the minimum possible volume.

The action of Mr. Meem's almost classical sand-box experiment is easily explained by the wedge theory. The construction is such that the granular material must act as a unit. Consider the effect on the two 48 by 12-in. sides of the box, and disregard (for the time being) the effect on the ends. Assume, for example, that the material weighs 100 lb. per cu. ft., has an angle of internal resistance of 40°, and that the angle of wall friction is 30 degrees. The width of the box is such that each of its sides may be considered as a retaining wall holding a wedge of sand, and, for simplicity of computation, assume that the two planes of rupture, starting on opposite sides of the box, intersect at the surface. For the dimensions given, this will be very nearly true. In addition to its own weight, each wedge carries half the material below the planes of rupture, the load being carried satisfactorily by the hanger bolts and plate washers. As the box does not fail, rupture is to occur in the fill; this is a case of passive resistance to pressure, which is, per foot of wall,

$$H \text{ (horizontal component)} = \tfrac{1}{2}yh^2 \tan^2 \tfrac{1}{2}(90° + \phi), \text{ or } 230 \text{ lb.}$$

The friction along the inner face of the sides is 230 (tan 30°) or 133 lb. per foot of length, and the total load to be held is only 50 lb., half the width of the sand. The effect of the ends of the box is to increase the factor of safety. Much depends on the tightness and rigidity of the box, and on the absence of vibration. In one of Mr. Meem's earlier papers,* a large-sized prototype of this model was tested, several men standing on top of the sand without failure, but, later, a slight jar broke up the state of equilibrium and the mass fell. Such a jar or shock temporarily loosens the contact along part of the side and prevents the action of the sides against the fill, hence, the passive resistance of the fill disappears, with a simultaneous drop in the total friction along the side. If this drop is great enough, and the total friction becomes less than the total weight, failure occurs.

Mr. Meem objects to the relative dimensions of the writer's apparatus, on the ground that the width should be much greater than the height. The writer agrees that much more accurate results could be obtained by using a wider bin, and, as Mr. Meem states: "Any measurements should include the whole area, as nothing can be gained by measuring the pressures of fractional areas, separately."

The writer would suggest, that, with a 6-ft. test wall, the bin should be 18 ft. wide, and the pressures on only the middle 6-ft. part should be measured. There would still remain the important problem, namely, the determination of the effect of the immovable wall, on each side of the test wall, on the recorded pressures. Testing the total pressure on walls much wider than that of the writer (5 ft.) will involve such problems as: Deflections of the wall, large pressure readings, and methods of support. The work entailed in using such an apparatus would require expensive machinery, and the cost would be prohibitive, unless backed by a wealthy corporation. No individual or university could be expected to undertake work on such a scale. Hence,

* *Transactions*, Am. Soc. C. E. Vol. LXX (1910), pp. 352–411.

the present design of relieving the friction along the sides, by using plates on bushed rollers, was adopted. During the preliminary tests, the effect of these plates—the most uncertain feature of the apparatus—was studied carefully. Leaving the plates "floating" in air, that is, perfectly counterbalanced vertically, rolling along hard steel cylinders immersed in oil along the plane of rupture, and bearing against smooth faces on the inner face of the concrete bin walls was finally considered the proper solution. As pointed out by Mr. Steinman, on page 1516, this design is open to the serious objection that a wedge of rupture is being assumed, and if any other shape of the mass tends to slip, the friction along the sides will overcome this tendency and influence the recorded pressures.

The force of this objection was realized, and its value was tested. The steel plates were overbalanced and, later, fixed in place, forming as rigid a part of the bin walls as the timbered face of the remainder of the wall. The results of the tests, with vertical wooden walls and horizontal fills, were only slightly less than the other results. As such a difference (about 5%) could easily be caused by a slight variation in the properties of the filling material, it was disregarded. This conclusion permitted the testing of oblique walls, in which tests the friction-reducing plates were not used. The small difference between the results of the tests with and without the side friction relieving plates makes the writer believe that, even with the 6-ft. height and a 5-ft. width, there was no appreciable arching action in the material tested.

In reference to Mr. Meem's suggestion of an apparatus for measuring the lateral earth pressure, Figs. 8 and 9, the writer sees no advantages in the method of counterbalancing pressures by the devices shown, and there are several disadvantages. Part of the measuring device is inside the bin which is to be filled. The removal of these parts to a point back of the bin, which is absolutely necessary in order to prevent interference with the fill, would require the use of long connecting strings or wires under high stress through the fill. The elongations will cause trouble in controlling the movements of the various parts of the wall; and there is no way of determining the effect that such wires (or rods, if members of larger section are used in order to cut down the deformations) will have inside the fill. Such a method has been used on a small scale to determine the overturning effect; it cannot measure the vertical component, directly.*

In describing the action of water-bearing soil, Mr. Meem states:

"The water exerts pressure through the voids of the soil, and finds necessarily reduced areas on which to exert its full head or pressure".

The writer can see no justification for this statement. Theoretically, it contradicts the very basis of mechanics, the third law of Newton: "To every action there is an equal and opposite reaction". Suppose that a wall is exposed to the pressure of a material having 40% voids immersed under a definite head of water. Then, in the average section through the mass, about 54% of the area is touched by water and 46% is in direct contact with the sand. The

* Engesser, "Neuere Versuche über die Richtung und Grösse des Erddrucks gegen Stützwände", 1893, *Deutsche Bauzeitung*, Vol. 27, pp. 325–326; Winkler, "Versuche über den Bodendruck", 1865, *Civilingenieur*, Vol. 11, pp. 1–11.

grains touching the wall are exposed to hydrostatic pressure, which acts in all directions. The sum of all the components, in a direction normal to the wall, is equal to the 46% which does not act on the wall directly. The grains take up this 46 per cent. On what do they re-act? The answer is obvious, as they are in direct contact with the wall. The grains touching the wall, as far as the hydrostatic pressure is concerned, merely act as part of the wall. The total pressure acting on the wall is the total hydrostatic pressure plus that due to the filling material, proper account being taken of the effect of the water on the unit weight and on the other characteristics of the material.

Experimentally, there are the tests of the pressure of concrete in forms. The work of Messrs. Shunk and E. B. Smith* has proved conclusively that the pressure of fresh mixed concrete is that of a liquid having a density of 150 lb. per cu. ft. In a perfect mix with zero voids, what should the pressure be, according to Mr. Meem's theory?

The writer agrees, in general, with Mr. Haines' statements, and perhaps can elaborate on some of them, tending to a more complete explanation. The angle of internal friction—in lateral earth pressure theories often called the angle of repose—does appear in all the expressions for the phenomenon of failure; and rightly so. The writer firmly believes that this factor is the angle of internal shear; and, in the description of his tests, he used a measure of this angle, the coefficient of internal resistance to shear. Failure in solids, granular materials, and liquids, when such failure occurs in shear, may be expressed in terms of an angle of internal shear, which is constant under identical conditions. The failure of concrete is explained by Considère by this method.† The writer described the failure of rocks under pressure, as expressed in terms of this angle, in his discussion‡ of the paper by Charles P. Dunn, Assoc. M. Am. Soc. C. E., on "Elastic Stresses in the Rock Surrounding Pressure Tunnels". An analysis of failure by the mathematical theory of elasticity shows that under stress there exists in a body a "shear cone", known as Lame's "cone of shearing stress", which is asymptotic to the ellipsoid of stress. The angle of the cone is a constant for any material under constant conditions, and for an elastic body under all conditions (up to the point of elastic limit). The variation of the angle of shear for granular materials has not yet been studied. It is probably influenced by such factors as pressure and temperature.

As to the arching effect in bracing and timbering cuts, Mr. Wiggin (pages 1522-1524), has voiced the opinions of the writer, and there is no necessity of repeating them.

Mr. McPike has brought up (page 1517) the question of the variations in the apparatus due to temperature changes, and the magnitude of movement required to record the pressures. The magnitude of the movements required to record the pressures has been described on page 1461, and Table 1 shows that

* Shunk, "Pressure of Concrete in Forms", 1909, *Professional Memoirs*, Vol. 1, pp. 247-260; Smith, "Pressure of Concrete in Forms", *Proceedings*, Am. Concrete Inst., 1920; Paaswell, "Pressure of Concrete in Forms", *Engineering and Contracting*, 1920, pp. 208–209.

† "Reinforced Concrete", 1906, translated by Moisseiff, see Appendix C on "Compressive Resistance of Solid Bodies".

‡ See p. 1398.

the recorded pressure is not dependent on the extent of the movement, within the limits of $\frac{1}{8}\frac{1}{000}$ and $\frac{1}{2}\frac{1}{000}$ in. vertically, and half of these limits horizontally. The writer has shown that the variations in the apparatus itself, due to temperature changes, were negligible. This is to be expected. The wall itself has no fixed restraints. Changes in temperature, of course, cause changes in the shape of parts of the apparatus; but no change in scale-reading is recorded, because the scales are not affected by temperature changes. If the wall was fixed, a different condition of affairs would exist.

Tests of the pressure of water-bearing soils can easily be made with the apparatus. As shown in Fig. 1, drain pipes were left in the bottom of the bin. It was intended to connect these pipes to a water tank, force water through the bottom of the fill, and determine the effect of such flow on the pressure. The apparatus is still at the University of Cincinnati, and it is hoped that these tests will be made in the near future.

Mr. Steinman has inquired about the anomaly in the conclusions, where the Coulomb theory, which is based on an incorrect assumption of normal pressure, fits the results of the tests better than the general wedge theory, which embodies the correct assumption of inclined pressure, for the case of a vertical wall.

The writer realizes that this anomaly exists and has no complete explanation. An inspection of the data will show that the difference between the two theories is small, and that either theory may be used to determine the magnitude of the resultant with only a small error. Two objections may be cited to show why the general wedge theory does not hold for all cases. The experimental researches in the subject have shown that the normal component of the pressure is independent of the type of wall used, that is, of the wall friction. Yet, the general wedge-theory formulas include the angle of wall friction in the evaluation of the pressure. The assumption of a solid wedge of rupture entails an appreciable error for the cases of large slopes, where the top of the wedge has a small angle. In such cases, the surface of rupture curves upward slightly and eliminates part of the wedge, causing pressure, The theoretical values, therefore, are too large.

Hummel and Finnan* obtained the same conclusions from their experimental determination of the pressure intensities in a mass of sand, using electric carbon resistance piles. For a vertical wall, they give the coefficients in Table 41.

TABLE 41.—COEFFICIENTS OF $\frac{1}{2}yh^2$.

ϵ	$-35°$	$-30°$	$-20°$	$-10°$	0°	10°	20°	30°	35°
Rankine theory	.....	.....	.....	.....	0.270	0.276	0.302	0.383	0.672
Wedge theory	0.202	0.214	0.230	0.250	0.270	0.301	0.339	0.437	0.677
Authors' formula	0.200	0.208	0.228	0.248	0.271	0.298	0.334	0.388	0.425
Authors' tests	0.214	.....	.....	.....	0.270	.....	.....	.....	0.408

* *Minutes of Proceedings*, Inst. C. E., Vol. CCXII (1921), p. 369–392.

Their formula is :

$$H = \frac{\frac{1}{2} yh^2}{\tan^2 \frac{1}{2} (90° - \phi) \left(1 - \frac{\tan \varepsilon}{\tan \frac{1}{2} (90° - \phi)}\right)}.$$

The same result—lack of agreement at the high slopes—is shown very clearly in Fig. 6. The writer cannot claim any absolute accuracy for his conclusions, but the tests indicate that, of the existing theories covering all possible cases, the general wedge theory approximates experimental results more closely than any of the others, with the restriction that the normal component (for a vertical wall, the horizontal) is independent of the nature of the wall. The results are incomplete, in that no tests were made with other than wooden-backed sloping walls; but the close agreement of the normal-pressure wedge theory in giving the horizontal component of the pressure in all the vertical wall tests is noteworthy.

Mr. Godfrey's fourteen points epitomize well the facts of the retaining wall problem. The natural (or should it be said, the logical) corollary is certainly warranted, and should be used as a criterion for the choice of the proper theory. He states:

"The only consistent theory for the design of retaining walls is one that approximates the results of tests and at the same time produces a wall corresponding in stability with successful walls."

His next statement, however, that "the simplest method of design is the equivalent fluid pressure method", is open to discussion. The writer's objections to this method are based on the grounds that it is not equivalent and that it has no right to the name "fluid".

The criteria for this method, quoting from the corollary, are the approximation of tests and the correspondence in stability with successful walls. That the method does not correspond with tests is evident. The equivalent fluid pressure method does not account for a vertical component, and the magnitude of the horizontal component, as noted by Mr. Godfrey, should be 0.17 ($\frac{1}{2} yh^2$), when the test gives 0.264 ($\frac{1}{2} yh^2$). The only reason that the equivalent fluid pressure method agrees with the stability of any existing wall is that the coefficient is chosen so as to give such agreement, the vertical component of the pressure being omitted in the analysis.

The advantage of the equivalent fluid pressure method, according to Mr. Godfrey, is the greater facility in computation and checking. He states: "The coefficient for the equivalent fluid pressure is all that is required in the solution of a problem in design, where there is no surcharge."

In one of Mr. Godfrey's former articles,* the method for the determination of this coefficient is the formula: Coefficient of ($\frac{1}{2} yh^2$) is $\tan^2 \frac{1}{2} (90° - \phi)$. Substituting, there results the ordinary Coulomb formula for the horizontal pressure, from the application of the wedge theory. In other words, the equivalent fluid pressure method consists of the use of the wedge theory to determine a coefficient, and then the use of this coefficient as the proper factor for the case under consideration, according to the simpler method. The writer

* "Design of Reinforced Concrete Retaining Walls", *Engineering News*, Vol. 56 (1906), pp. 402–403.

prefers to use the wedge theory "straight". In his discussion, Mr. Godfrey gives this method in substance:

"The equivalent fluid pressure method of design is not restricted to any theory, as it can be made to give results identical with the theories mentioned by Mr. Feld, merely by using a suitable coefficient."

Should he not have added: "which coefficient is determined by the theory"?

Does Mr. Godfrey imply that the use of the theories should be restricted to a select group of engineers who are to issue "coefficients" for the use of the ordinary designer? He states that "it is true, also, that a vertical force is seldom considered in a design, and for safety should be neglected." In practically every retaining wall on an ordinary foundation, the criterion governing the width of the base is the requirement that there shall be no tension at the heel of the wall, and that the maximum allowable soil load must not be exceeded at the toe. The eccentricity of the resultant pressure on the foundation is caused largely by the overturning effect of the horizontal component. Taking the vertical component into account—and there is no doubt that a vertical component exists—decreases this eccentricity as well as the overturning moment. A proper design, of uniform strength and with a known factor of safety, can then be made. In walls resting on pile foundations, the vertical component can hardly be disregarded—the difference in design is too large. In bins for storage, where a large part of the filling material is carried by the walls, the friction cannot be disregarded. As the foundation would not be strong enough; the vertical component certainly cannot be disregarded for the "sake of safety". As Mr. Paaswell has stated: "Examine the records of the failures of retaining walls, and note how few collapses are due to overturning moments". And, the writer would like to add: Examine the records of successful walls in Europe, and note how much lighter the sections are when compared with the usual American design. Considering present costs of construction, it is about time that the designer realized the waste of materials and labor to which he is a party. This statement is made in spite of the failures which occur from time to time, as a careful examination will invariably show that there was too much material at all points except one, which was probably "too difficult to compute", and so it was "guessed". It would have been cheaper to construct a properly designed structure which would not have failed.

After comparing the results shown in Table 5, and the coefficients as given by the equivalent fluid pressure method, Mr. Godfrey states:

"The conclusion, therefore, is that in the design of retaining walls to sustain earth fill, the pressure of sand is greater than need be provided; and this is what is to be expected because of the cohesion of the earth."

In order to compare the results of Table 5 with Mr. Godfrey's method, and his statement that a stone wall of a width one-third of its height is stable as a retaining wall, the writer made the following comparisons in design:

Assume an earth of 100 lb. per cu. ft., $\phi = 37°$, $\phi' = 32°$, retained by a vertical wall 20 ft. high, the fill being level with the top of the wall:

The horizontal component is 0.264 ($\frac{1}{2}yh^2$), or 5 280 lb. per ft.
The vertical component is 0.625 (5 280), or 3 300 lb. per ft.

Mr. Godfrey gives as the horizontal pressure: 0.17 ($\frac{1}{2}yh^2$), or 3 400 lb. per sq. ft. The equivalent fluid pressure method gives an overturning moment of 22 700 ft-lb. The resisting moment of a rectangular wall, with a base width one-third of the height and a density of 150 lb. per cu. ft., is 66 700 ft-lb., or a safety factor of 3 for overturning. As the resultant intersects the base just outside the one-third point, it seems that the coefficient was chosen to satisfy this condition. The writer's results indicate that the overturning moment was 17 600 ft-lb., a factor of safety of almost 4, and that the resultant intersects the base at 2.29 ft. from the toe, that is, this type of wall is quite stable and there is no tension at the heel.

The comparison of a trapezoidal section, having the same characteristics as in the previous case, vertical back, and top width of 2 ft., is: The resisting moment now is 55 750 ft-lb., with the center of gravity of the wall 4.29 ft. from the toe. Mr. Godfrey's method shows a factor of safety against overturning of 2.5; and the resultant intersects the base 2.54 ft. from the toe (well inside the third point).

The writer's results show that the factor of safety against overturning for this wall is 3.2, and the resultant intersects the base at 2.60 ft. from the toe. It is very evident that the coefficients of the "equivalent fluid pressure method" are derived either from the wedge theory or from walls designed by the wedge theory. The writer, therefore, can see no reason for the name given these coefficients. Mr. Godfrey states that: "In next to the last column of Table 5, values are given that could be used as a coefficient for an equivalent fluid pressure." As he realizes, however, these coefficients are too high for his method and would require needlessly wide walls; the vertical component must also be taken into account.

Mr. Godfrey notes that the differences in readings of the corresponding scales in Table 3 discredit the exactness of the lateral pressure theory. The reasoning is most illogical. There was no contract between the theory and the apparatus that they should agree; at least, the writer knows of no such contract. Such differences in the horizontal pressure at the two lower corners of the wall have absolutely nothing to do with the theories. The wall is acting as a stiff plate resting on three horizontal contacts. If one contact was slightly stiffer than the other, that is, in adjusting the apparatus preliminary to the test, one contact had not been loosened as much as the other, it would take a larger part of the load.* This happened several times; at first, the test would be repeated after the apparatus had been re-adjusted. It was found that the sum of the readings remained the same, and as it was the sum that was required, the inconsistency in readings had no effect on the results. This was also true of the recorded vertical pressures.

The writer agrees with Mr. Godfrey that the tests with superimposed loadings are neither complete nor decisive; however, the results obtained will be found to be quite in agreement with each other. There is undoubtedly an

* See "Method of Operation," p. 1458.

increase in pressure when loads are placed outside the theoretical wedge of rupture, but the amount of such increase warrants the conclusion on page 1491, that the increase may be disregarded. The present theory assumes that a discontinuity exists at the point where the surface of rupture cuts the top surface of the fill. The tests indicate that such a discontinuity does not exist, but is a close approximation to the truth.

Mr. Godfrey's problem is too indefinite to allow a solution; for, as he says, "the designer wants to know at what distance from a retaining wall a railroad track will exert pressure against the wall." As very few designers would try to solve the problem unless given the height of the proposed wall and the distance from the wall to the center line of the track, the writer hopes to be pardoned for not answering Mr. Godfrey's question. If the method he gives is supposed to be applicable to all heights and all locations of the track relative to the wall, the writer does not hesitate to go on record as being strongly opposed to it.

The suggestion by Mr. Godfrey, that the experimental results for the pressures of oblique fills against vertical walls be compared with his method, in order to ascertain "whether one-half the vertical height of the triangle of surcharge added to h will not give the horizontal pressure of his tests with about the same coefficient as that found without surcharge", resulted in the writer computing Table 42, as supplementary to Table 8, in an attempt to determine the existence of such correlation. The test was with a vertical wooden wall, the height of fill at the wall was 5 ft., and the slopes of the surface are indicated in Column 1. It was assumed that the plane of rupture bisects the angle between the plane of repose and the wall, which is approximately correct. In Column 2, h' denotes the height of the highest point of the wedge of rupture, above the wall:

$$h' = \frac{h \,.\, \tan \varepsilon}{\tan \tfrac{1}{2} (90° + \phi) - \tan \varepsilon}.$$

Column 3 gives e, Mr. Godfrey's equivalent height, or $h + \frac{1}{2} h'$. As the total pressures vary as the square of the height (the conditions and, therefore, the coefficients being the same), the coefficients for Mr. Godfrey's method, c in Column 4, are equal to the ratio, $e^2 : h^2$ multiplied by the coefficient for the case, $\varepsilon = 0$, $h = 5$ ft., that is, $0.237 \frac{e^2}{h^2}$. Column 5 gives the experimental coefficients for the horizontal components. It may be seen that the average difference between experiment and Mr. Godfrey's method is twice as large as that between experiment and the wedge theory.

Mr. Wiggin states that the test wall being "supported rigidly at top and bottom, cannot entirely represent the ordinary wall. The Goldbeck cell for measuring pressures in earth offers a better means than was formerly available for observing pressures and changes of pressure with lapse of time in earth back of actual structures." The test wall was, in reality, a "free body" with motion in two directions restricted, but not prevented. The readings were recorded before the maximum movement possible was attained. The

wall was free to translate horizontally or vertically and to rotate about the bottom (tipping) or the top (settling at the heel). The kind of movement which really occurred (very small in amount) depended on the nature of the fill.

TABLE 42.

(Supplementary to Table 8).

ϵ (1)	a'. (2)	e. (3)	c. (4)	(C)—Experimental. (5)
−33° 40′	−1.28	4.36	0.180	0.179
−31°	−1.18	4.41	0.185	0.179
−28° 05′	−1.06	4.47	0.190	0.177
−25°	−0.970	4.52	0.195	0.177
−21° 50′	−0.855	4.57	0.198	0.179
−18° 30′	−0.740	4.63	0.204	0.184
−14° 55′	−0.600	4.70	0.210	0.185
−11° 20′	−0.465	4.77	0.216	0.193
− 7° 40′	−0.325	4.84	0.223	0.204
− 3° 50′	−0.165	4.92	0.230	0.221
0 00	0.000	5.00	0.237	0.237
3° 50′	0.180	5.09	0.246	0.250
7° 40′	0.375	5.19	0.256	0.260
11° 20′	0.575	5.29	0.266	0.282
14° 55′	0.795	5.40	0.277	0.288
18° 30′	1.04	5.52	0.290	0.308
21° 50′	1.30	5.65	0.302	0.327
25°	1.57	5.78	0.317	0.351
28° 05′	1.90	5.95	0.335	0.391
31°	2.24	6.12	0.355	0.395
34°	2.66	6.33	0.380	0.423

The writer is in full agreement with Mr. Wiggin relative to the accuracy and sensitiveness of the Goldbeck pressure cell, but doubts the applicability of this apparatus to the problem discussed in the paper. The objections to the pressure cell are: The small area exposed to pressure, and the fact that the readings are obtained after a movement of the disk against the fill has been accomplished. The latter objection, because of the very small size of such movement, is not as important as the former. A filling material is not homogeneous, and a pressure cell can only record the pressure at its face, which pressure may be far from the average condition existing in the fill.

It may be of interest to know that the writer had planned to insert several of these pressure cells in the face of the test wall. As the University of Cincinnati had no funds available for this purpose, an attempt was made to borrow some cells from the U. S. Bureau of Public Roads. The Bureau was most kind in its offer of several cells when they should become available; but, unfortunately, that was after most of the tests had been completed. The writer hopes that this part of the plan will be carried out in future tests at the University.

Mr. Wiggin's explanation of Mr. Meem's observations in timbered cuts can be extended by the following example: Assuming the truth of the wedge

theory, for a material of such nature that the wedge will stick together as a unit, it can be shown that, after a certain ratio of depth to width is exceeded, the earth, as in a pit, is self-supporting. The same phenomenon occurs in a braced cut, where a part of the cut is exposed and the material on each side is well braced and supported.

The horizontal pressure of the material, or the horizontal reaction which is required to prevent slipping, is:

$$\tfrac{1}{2}\, y\, b\, h^2 \tan^2 \tfrac{1}{2}\,(90^\circ - \phi) \qquad (a)$$

the unit pressure at the depth, x, is:

$$y\, x \tan^2 \tfrac{1}{2}\,(90^\circ - \phi) \qquad (b)$$

The writer is assuming a horizontal fill and a vertical wall, of width, b, and height, h. The wedge tends to move, as a unit, parallel to the plane of rupture, an angle of $\frac{1}{2}\,(90^\circ - \phi)$ to the vertical, and is resisted by the friction along the two triangular sides of the wedge. The friction along the plane of rupture has been considered in the evaluation of the required balancing force. The total pressure on each side of the wedge, the width, s, of a strip, dx high, being,

$$(h - x) \tan \tfrac{1}{2}\,(90^\circ - \phi),$$

is,

$$\int_0^h yx\, s \tan^2 \tfrac{1}{2}\,(90^\circ - \phi)\, d\, x$$

or,

$$\left[\frac{yh^3 \tan^3 \tfrac{1}{2}\,(90^\circ - \phi)}{6}\right] \qquad (c)$$

Acting parallel to the direction of motion, $\frac{1}{2}\,(90^\circ - \phi)$ to the vertical, there are frictional resistances on the two sides, the total value being,

$$\tan \phi \left(\frac{y\, h^3}{3} \tan^3 \tfrac{1}{2}\,(90^\circ - \phi)\right) \qquad (d)$$

The horizontal component of this resistance is:

$$\tfrac{1}{3}\, y\, h^3 \tan \phi \tan^3 \tfrac{1}{2}\,(90^\circ - \phi) \sin \tfrac{1}{2}\,(90^\circ - \phi) \qquad (e)$$

For equilibrium, Equations (a) and (e) must be equal, from which results the equation:

$$b = \tfrac{2}{3}\, h \tan \phi \tan \tfrac{1}{2}\,(90^\circ - \phi) \sin (90^\circ - \phi),$$

as the relation between b and h for the case of zero horizontal pressure. From this, Table 43 was computed.

A similar investigation shows that the requirement for zero vertical pressure is:

$$b = \frac{2}{3} \frac{h \tan \phi}{\tan \phi'} \sin \tfrac{1}{2}\,(90^\circ - \phi),$$

and assuming that $\phi = \phi'$, there results the values in Table 44.

The cohesion has been disregarded as its effect is to increase the resistance and decrease the pressure. It should be noted that the maximum condition occurs for zero horizontal pressure when ϕ is about 35°, when a vertical cut,

0.112 as wide as high, is self-supporting, without taking into account the action of cohesion. For larger or smaller values of the angle of internal resistance, the width must be a smaller part of the height.

TABLE 43.—VALUES OF $b:h$ FOR ZERO HORIZONTAL PRESSURE.

ϕ	0	5	10	15	20	25	30	35	40
$\frac{b}{h}$	0.0	0.036	0.065	0.084	0.098	0.106	0.111	0.112	0.110

ϕ	45	50	55	60	65	70	75	80	85	90
$\frac{b}{h}$	0.106	0.099	0.090	0.080	0.069	0.056	0.043	0.028	0.016	0.00

The writer agrees with Professor Menefee's discussion in every point. The use of the term, "elasticity", in connection with soils is not entirely objectionable, if by soils (as defined by the writer in the beginning of this discussion) is meant all the materials which cannot be classed as solid or fluid. The writer was unable to find a better term than elasticity to cover that factor, on which soil action depends and which corresponds to the "solidity" and elastic strain of the material. The writer, however, does not include the requirement of Hooke's law in the term, "solidity."

TABLE 44.

ϕ	0	5	10	15	20	25	30	35	40
$\frac{b}{h}$	0.471	0.450	0.428	0.406	0.382	0.358	0.333	0.308	0.282

ϕ	45	50	55	60	65	70	75	80	85	90
$\frac{b}{h}$	0.256	0.228	0.201	0.173	0.144	0.116	0.087	0.056	0.029	0.0

Professor Cain deserves the thanks of the Society for his painstaking analytical discussion, and the writer feels highly complimented to receive it. A discussion of that nature from the man who may rightly be called the Dean of the Soil Physicists of the country, proves to the writer that his efforts to contribute a share to the complete solution of the problem have not been in vain.

The conclusions for the effect of wall friction would certainly have a better experimental foundation if there was a greater variation in the coefficient of wall friction. The variation of the friction of sand or earth on the materials generally used for wall backing, however, is surprisingly small. Polished sheet metal showed the smallest coefficient; the coefficient with glass was larger, a rather unexpected result. The minimum value, 24° 40′, is

not far from the maximum possible value, the angle of internal resistance, because if ϕ' should become greater than ϕ, a thin layer of earth will attach itself to the face of the wall, and any displacement which may occur will be along a surface bounded on both sides by earth, as there is a less frictional resistance on such a surface than along the back of the wall. This is clearly pointed out by Professor Cain in all his papers and books, yet he seems to have overlooked it, as in five places in his discussion there appears the case of ϕ equal to 30° and ϕ' equal to 32° 30′. These assumptions are made to show a closer correlation between theory and experiment. The writer cannot consider any conclusions to be drawn from such an assumption.

The detailed data on the determination of the coefficient of wall friction may be of interest. The method used is described on page 1462. The height of the sand in the box was equivalent to a pressure of about 26 lb. per sq. ft. on the area along which the friction was being measured. For dry wood (yellow pine, across the grain), the average of nine readings, for a moisture content of 9.4%, gave a coefficient of 0.492 (corresponding to an angle of 26° 10′), with a maximum variation of 10%; for the wet wood, the corresponding figures were 0.585 (30° 30′). In these tests, there was little variation between the static and kinetic friction.

For sheet metal, the static friction was 0.530 (28° 00′) and the kinetic friction was 0.460 (24° 40′). For plate glass, the static friction was 0.535 (28° 10′) and the kinetic friction was 0.505 (26° 50′), the maximum variation being 7 per cent. Although far from conclusive, the tests indicate a surprisingly small variation. The writer expects soon to continue this phase of the problem.

The effect of the water content, with the usual type of filling material, the writer believes will be found not to be as great as is expected, although he has no data to prove it. Lubricating action will only occur in those materials which contain a rather high colloid content, such as loams and clays, and on those which are changed, either chemically or physically, by the action of water, such as shales and marls.

The writer agrees with Professor Cain that the accurate determination of ϕ' is more important than that of ϕ, as the magnitude of the pressure varies only slightly with a variation in ϕ, whereas the magnitude of the overturning moment and also of the distribution of the pressure on the foundation, is greatly affected by ϕ'.

In reference to the statement on page 1474, that the plane of rupture bisects the angle between the plane of repose and the wall, the writer fully realizes that it has no theoretical support. In spite of this, the statement has been made in several standard books, and the writer included the computations based on this assumption to show that it also had no experimental basis. Some authors even make the statement that the plane of rupture bisects the angle between the plane of repose and a sloped wall.*

In a recent discussion of this method of evaluating the lateral pressure, the writer pointed out some inconsistencies which exist in the wedge theory,

* Hool and Johnson, "Concrete Engineers' Handbook", 1919, p. 579.

and that this assumption is not the proper solution to avoid these inconsistencies, as is claimed by some.* As, theoretically, the area of the wedge of rupture becomes infinite as the surface slope approaches a maximum, a correlation between the acting pressure and the area of the theoretical wedge is useless. It would be desirable to have a simple relation between the pressure and the assumed theoretical wedge (that is, the plane of rupture bisects the angle between the wall and the plane of repose), but no such relation exists. The writer attempted to derive an empirical formula, of the straight-line, power, or exponential form, to show the relation between pressure and the angle of surface slope, and a similar formula for the relation between pressure and the wall angle, but it was found that such formulas were more complicated than theoretical wedge formulas.

The following data concerning the physical properties of the material may answer, partly at least, Professor Cain's inquiry relative to the methods used in determining the various constants. As a general rule, two samples were taken from each test, one at about the middle and one at the end, the latter being the material at the surface. The physical determinations were made on a mixture of these two samples. The weight per cubic foot of the material, as shoveled into the air-tight sample cans, was found for the loose and the packed conditions, and the average value was used in the computations. At first, the method was to weigh the full can of earth as shoveled in, compact the material by shaking and jarring, and then determine the volume of the loose and the packed material by weighing equal volumes of water. The weight of the empty can completed the required data for the determination of all the possible information concerning the unit weight. Lack of time prevented the continuation of this procedure in the later tests, in which only the average volume was measured. The writer could get no one to aid him in this very uninteresting routine work and had to resort to this short-cut method for obtaining the average values in order to make available the sample cans for other tests. As the main apparatus was not well protected, the tests had to be completed before cold weather.

The most complete determinations were for the materials used in the sheet metal and glass walls (Tests Nos. 3 and 4), as given in Table 45.

TABLE 45.

Test No.	Percentage of moisture.	Unit Weight, in Pounds per Cubic Foot.		Average weight, in pounds per cubic foot.
		Loose.	Packed.	
3	4.9	89.	97	93
4	4.2	89.5	99	94

A summary of all the data is given in the description under Test No. 1. The detailed description was not included in the paper, because the author did not feel that it was complete, but as it may be of use or of interest to

* *Engineering News-Record*, March 22, 1923, p. 553.

some of the members, it is now given. For the friction tests, the box used had a cross-sectional area of 24 sq. in. and was of very light construction. In two cases, the coefficient was determined under varying load on the box. Fig. 28 shows very clearly that there was no cohesion. The plotting of Professor Cain's formula, $q = f\ p_n + c$, passes very close to the origin. These tests, however, were determined by the spring-scale method, and probably do not include the total resistance to shear, and therefore, the writer does not consider them to be conclusive. The straight-line variation seems to hold over a rather wide range.

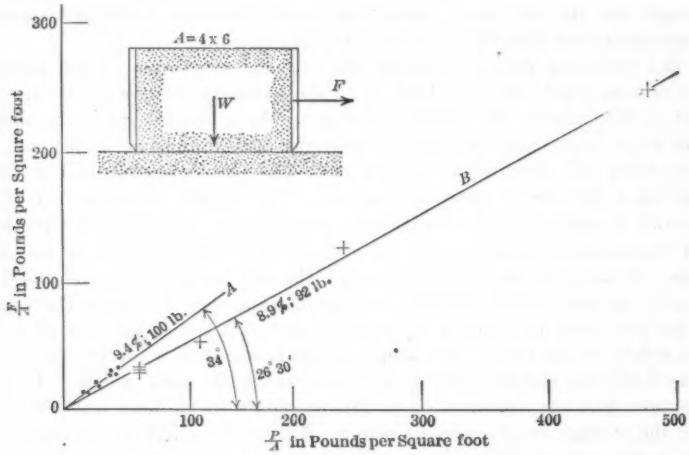


FIG. 28.

Table 46 shows very clearly the relation between the unit weight and the moisture content. The relation between the moisture content and the various coefficients is not quite so evident. The reason for the lack of data with a moisture content between 8.9 and 4.9% is that the material dried out partly during the summer. In the fall, it was shoveled several times before a uniform distribution of moisture, as shown by the color, was obtained.

The moisture content is by weight; about 15-lb. specimens were weighed directly after the removal of the material from the specimen cans and then about a week later, during which period the material had been spread out on a stone table exposed to the sun for several hours each day, in a layer about 1 in. thick. The moisture content for Tests Nos. 5 and 6 was 8.9% and for Test No. 7, 4.2 per cent. These data were accidentally omitted from the tables.

As shown by Table 2, the materials at the top, at 6-ft. and at 10-ft. depths, differ only slightly from each other. The three boxes were somewhat more than 5 in. on each edge, holding about 12½ lb. of earth. It is probable that the moisture content and unit weight are constant throughout the greater part of the mass, with the exception of the material near the surface. As is well

known, the voids in a soil decrease to a minimum value as water is added; additional water causes an increase again until the soil is saturated. With minimum voids, the unit weight is a maximum. The writer believes that, under usual conditions, moisture will distribute itself so that the minimum voids will exist at all points. Excess water will seep through; insufficient water will be unevenly distributed, from zero at the surface to the value which causes minimum voids at a short distance below the surface. In the latter case, the distribution will not be in accordance with a straight-line variation. In other words, soil tends to arrange itself in the smallest possible volume.

TABLE 46.—PHYSICAL PROPERTIES OF THE FILLING MATERIAL.

Percentage of moisture.	Weight, in pounds per cubic feet.	Angle of natural slope.	Angle of internal resistance.	Angle of internal friction.
0.0	103	30° 15′	28° 20′	28° 20′
3.05	93	35° 00′	31° 20′	29° 30′
3.24	94	38° 20′	31° 50′	28° 30′
3.49	96	32° 45′	29° 40′	27° 30′
3.50	97	34° 00′	31° 00′	29° 00′
3.55	97	36° 30′	30° 20′	27° 30′
3.65	97	34° 50′	30° 30′	27° 55′
3.73	97	35° 00′	31° 00′	29° 15′
3.73	97	36° 00′	31° 50′	29° 40′
4.2	94	38° 00′	34° 30′	32° 00′
4.9	93	35° 30′	33° 50′	31° 50′
8.9	95	39° 30′	38° 00′	35° 30′
9.4	100	40° 00′	37° 00′	34° 00′

With reference to Professor Cain's description of a different method for the determination of the coefficient of internal friction, the writer believes that a refinement of his own simple apparatus will give more consistent results with less work. A shear surface in the material is all that is required, the resistance along which can be measured with as little difficulty as possible. The open box with beveled sides acts as the normal pressure on the surface of the material bounded by the base of the box. The box may be filled to any height and loads placed on top of the box, or on the material, for the heavier unit pressures. These loads, if resting directly on the box, are transmitted to the earth by the frictional shear resistance along the sides. Too large a load placed in this manner will cause the box to dig into the lower surface of earth, which is objectionable and must be guarded against.

In describing Mr Griffith's apparatus, Professor Cain states:

"The weight thus transmitted to the sides of the upper box can be ascertained, perhaps with sufficient accuracy, by measuring the vertical force necessary to draw the upper box upward and away from the sand."

The writer doubts very much whether this vertical force subtracted from the total weight of the upper box will give the load bearing on the surface along which the friction is being measured. In the writer's method, the entire weight of the upper box is the load, and no auxiliary data are required.

It is rather unfortunate that Professor Cain assumed ϕ' greater than ϕ, in attempting to show a closer agreement between theory and experiment. It must also be remembered that any set of data can be shown to agree exactly with any theory if one or more of the series of figures obtained is changed to suit such agreement; such methods are open to serious criticism.

Before any tests were performed with sloping walls, a vertical false wall was attached directly to the back of the main wall, in order to determine the effect of the eccentric loads, the false wall, and the vertical component, on the method of measuring the pressures. This test was made with the dryer type of material, and completes the set of tests with the moisture content about 4 per cent. It was omitted from the paper, because it was not performed with such great care as the other tests and because it gave no conclusions not covered by previous tests. It is given in Table 47.

TABLE 47.—TEST NO. 7*a*, PRESSURE AGAINST A VERTICAL FALSE WALL.*

Date: October 25, 1921.
Weather: Clear.
Temperature: 18° cent. (65° Fahr.).
$h = 4$ ft.; $\alpha = 0$.
Moisture content: 4.2%.

Physical Properties:
Weight per cubic foot = 94 lb.
Angle of Natural Slope = 38°30′
" " Internal Resistance = 37°35′
" " Internal Friction = 34°50′
" " Wall Friction = 28°

Surface slope, ϵ.	Pressures, in Pounds per Foot.			ϕ'	$\frac{x}{h}$	$\frac{H}{\frac{1}{2}\gamma h^2}$	J	H'
	H.	V.	P.					
00° 0	169	82	188	24°	0.336	0.225	0.240	0.194
4° 45′	174	83	193	25° 30′	0.370	0.232	0.253	0.205
9° 30′	179	88	199	26° 30′	0.372	0.238	0.264	0.215
14° 00′	185	98	218	27° 30′	0.380	0.246	0.280	0.230
18° 30′	200	107	227	28°	0.386	0.266	0.297	0.246
22° 30′	212	116	242	29°	0.393	0.282	0.313	0.263

* For the notation, see the explanation of Test No. 5, p. 1473. The conclusions to be drawn from Test No. 7*a* are the same as those drawn from Test No. 5.

Professor Ketchum raises two objections to the apparatus, namely, the small width and the necessity of movement of the wall to record pressures. As far as the possibility of arching is concerned, that has already been fully discussed. As to the necessity of motion, all present-day methods of determining stress are by a comparison of the strain with a fixed standard. The writer refers Professor Ketchum to some notes on page 1503 on this subject. "The pressure-measuring apparatus described in the writer's book is the only one known to him that is not subject to very severe criticism". The writer hoped that he had made clear the advantages of the platform scales over all indirect methods of measuring the pressure. The carbon piles mentioned by Professor Ketchum, must consistently and accurately record pressures of about 2 000 lb., and greater, if a wider bin is to be used in order to "eliminate the arching action". The writer studied the subject of carbon piles, and in addition to the objection that they must be calibrated before each test, he found that a unit, in order to record pressures of 2 000 lb., would have to be of large

cross-section if the maximum deformation is to be kept to the values which were obtained with platform scales. It may also be of interest to know that the writer investigated the possibility of using other methods of recording the pressures, such as elastic struts, plungers in oil, friction dynamometers, deflecting plates, etc., as well as the carbon and other better types of resistance piles and platform scales.

Professor Ketchum does not seem to be convinced that the frictional component along the back of the wall should be considered. The tests do not satisfy the conditions assumed by Rankine, in that the pressure-measuring surface must be sufficiently rigid to prevent movement of the mass. If any apparatus could be devised, which is so rigid that there is absolutely no deflection and, consequently, no movement of the mass, the writer agrees that no frictional resistance component would be found. He is also convinced that no other component would be found. In other words, if Rankine's assumptions are fulfilled, the wall is perfectly stable. A careful survey of the basis of all design will show that the investigation is at the point of non-stability, the factor of safety or the reduced unit stresses assuring that this point will not be reached. If an economical retaining wall is desired, it should be designed for the case of incipient failure under the worst conditions; the use of reduced unit stresses will then assure the safety of the structure. Under these conditions—and they are the actual conditions—the writer cannot see how Professor Ketchum can continue to use the Rankine assumptions for the analysis of retaining walls.

It may be of interest to know that as early as 1860, J. J. Sylvester,* one of the most noted mathematicians of the United States and England, made a careful analysis of the bases of the wedge and the Rankine theories. He states that Rankine had misused the Moseley principle of least resistance, which applies only to a system in equilibrium. The lateral pressure, in the Rankine theory, is determined at the state of incipient motion during which it cannot be assumed that the summation of the pressures is a minimum, as stated in the Moseley principle. Another objection is the use of the angle of internal friction as the inclination of the resultant at the point of non-equilibrium; although the inclination of the resultant at every point must be less than this angle, there is no basis for the assumption that the inclination is equal to this angle at the limiting point.

Professor Ketchum states that the writer has "erroneously assumed that the Rankine theory" is not general. The writer is well acquainted with the theory, as developed by Rankine, as well as the versions of several other men. By the Rankine theory, the writer means the Rankine theory and no version thereof. A letter from Rankine to the Secretary of the Royal Society (Professor Stokes), announcing his intention of writing a paper on the subject and stating his premises, has been published.† This letter was dated February 18, 1856. The paper was presented on June 19, 1856, before the Royal Society. It is very curious to note that on September 27, 1856, three months later, Carl Holtzmann's work entitled, "Ueber die Verteilung des Drucks im Inneren

* "On the Pressure of Earth on Revetments", 1860, *Philosophical Magazine*, pp. 489–499.
† In the *Philosophical Magazine*, Vol. II, 1856, pp. 469–470.

eines Koerpers", appeared in print.* The lateral pressure for the special case of vertical wall and horizontal fill is derived from the ellipsoids of stress of Cauchy and Lamé. The method is almost identical with that of Rankine, the applications of which are given in his "Civil Engineering" and "Mechanics". In "Civil Engineering",† he states:

"*Pressure of Earth Against a Vertical Plane.*—Consider a mass of earth, whose upper surface, *YOY*, is either horizontal or inclined at any angle, ϕ, and is cut by the vertical plane in a direction perpendicular to that of steepest declivity. * * *

"The *direction* of the resultant is already known to be parallel to the declivity, *YOY*."

The same statements are made in his "Applied Mechanics", and, in addition,‡ he shows a sloped wall and takes the pressure on a vertical plane through the heel. If the wall has negative batter, he states that the weight on the base is equal to the weight of the masonry in front of this vertical plane minus the weight of earth of a volume equal to the volume of masonry behind the plane. This is equivalent to the assumption of an upward pressure on the back of the wall. How can Professor Ketchum connect this method with the premise: "(*a*) There can be no active upward component in a granular mass"?

To summarize: As Rankine probably realized, and as used by him, the ellipse of stress method takes one of the conjugate stresses, the weight due to gravity, as vertical; hence, from the very definition of conjugate stresses, the other stress must act on a vertical plane. The writer, therefore, cannot see that he has omitted any possible comparison between his experimental results and the Rankine theory. Some work had been done to correlate the tests with the Weyrauch and Mohr theories, but the material was not sufficiently complete for presentation. The results obtained showed no agreement or conclusions of value.

The writer will repeat that he did not see any surface of rupture, nor did he expect to see any. The maximum movement of the wall was just outside of microscopic dimensions, and there was no microscope available. The writer admits the error in the statement concerning the plane of rupture, and refers to the preceding discussion for the reason. He was rather surprised to read the statement:

"The so-called plane of rupture bisects the angle between the vertical and the plane of internal friction only for a vertical wall with a horizontal surcharge, with the resultant pressure normal to the wall, as will be given by Rankine."

The writer has never seen any reference to a plane of rupture in Rankine's works. It would have been very interesting to learn some reasons for the last statement in Professor Ketchum's discussion.

On first reading the lengthy and detailed discussion of Dr. Terzaghi, the writer thought it was a valuable contribution to the subject. However, closer study of the argument advanced shows that it is based on three assumptions,

* *Einladungsschrift* der K. K. Polytechnischen Schule in Stuttgart.

† 23d Edition, p. 323.

‡ 18th Edition, p. 250.

one of which is entirely wrong, and at least one of the other two is very doubtful. The assumptions are:

(1) That the experiments performed by Dr. Terzaghi on a model wall, 10 cm. (4 in.) high, with a crushed sifted quartz averaging 0.1 in. in diameter, or one-fortieth of the total height of the wall, are absolutely accurate. The coefficient, 0.42, is, therefore, not to be doubted and holds for all the cases for the maximum pressure of the material behind the wall. The writer can hardly admit that the material of the Cincinnati tests was crushed sifted quartz.

(2) That the results of these miniature tests—"toy experiments" is a name given to this type of research in several places in earth pressure literature—are absolutely applicable to the results of tests with a 6-ft. wall, an exterpolation of 1800 per cent. This assumption is hardly an axiom, yet it has neither experimental nor theoretical proof.

(3) That the Cincinnati tests proved the existence of a straight-line variation relation between the acting pressure and the amount of wall movement. This is totally in error; Dr. Terzaghi misread the text in connection with Table 1 and did not use the proper data.

It must be remembered that Dr. Terzaghi had certain ideas fixed in his mind, and that he states that certain of the tests with vertical walls give an experimental proof to his theory. He makes no attempt to generalize his ideas to the cases of sloped walls or sloped fills, nor does he attempt to correlate most of the tests with his theory.

He states: "If the wall yields, the pressure should decrease rapidly and be a continuous function of the distance covered by the wall before the first 'slip' occurs." It will be shown subsequently how the very data he uses to prove this statement, actually prove just the opposite.

Dr. Terzaghi states that "the earth pressure acting against a perfectly rigid retaining wall should be of the order, $0.42 \frac{y h^2}{2}$." No wall is perfectly rigid; if it was, it would not fail under any pressure, no matter how large. "The coefficient, 0.42, was determined for crushed and sifted quartz and was found to be independent of whether the sand was loose or compacted." If it is remembered that 40 grains, of the sand used, made up the total height of his test wall, the value of the conclusion is immediately evident. It is not reasonable, and it also contradicts the other parts of his theory. For the "slip" in a loose material will occur much sooner—that is, with a less wall movement—than in the compacted material.

Dr. Terzaghi also states: "From the diagrams published by the writer, it may be learned that the first slip involves a total horizontal outward movement of the wall of about $0.001 h$." As it is safe to assume that the loose and the packed material exert the same pressure at the point of slip, this would require that the pressure of the packed material be less than that of the original loose material at the point where the wall has moved sufficiently to make it of the same nature as the loose material. The movement of the wall necessary to permit the grains of the packed material to assume the condition of the loose material is probably far more than $0.001 h$ (except for very large

walls), when there occurs the case of slip in the compacted material following a wall movement of 0.001 h, and while the material is not in the same condition as the loose material at its first slip:

"This process was called by the writer the 'first phase' of the earth pressure, and involves a decrease in lateral pressure of about 48% to 31% of the original value, according to the density of the structure of the sand."

In other words, the decrease in pressure from the original maximum to that at the point of slip depends on the "density of the structure", or the compactness of the original material, to bring the values to the same pressure at the point of slip. Hence, the original values could not be the same, and this also contradicts the statement of the infallibility of the coefficient, 0.42.

Dr. Terzaghi determines a value which he calls the "modulus of elasticity of sand". From the definition given in Hooke's law, the term, "Young's modulus of elasticity", denotes a constant. The writer, therefore, would prefer a different name for the values in Table 37. It is also noteworthy that these values have been obtained by compressing a mass of sand in which the lateral expansion has been prevented. This is in no way analogous to the similar property of solids; in determining the modulus of elasticity of a solid, lateral expansion does occur, as is explained by Poisson. Therefore, the values of s_1 in Table 37, based on the modulus of elasticity, also contain this error. Dr. Terzaghi omitted from his discussion an explanation of the real source of the values, s_1. The statement that a movement of 0.00483 cm. will decrease the pressure against a 6-ft. wall by 10% is open to criticism as it depends on a wrong value of the modulus.

Dr. Terzaghi also states that the movement required for the occurrence of the first slip is 0.001 h, which may have been true for a 4-in. wall, but that is no proof that it is true for a 72-in. wall. Incidentally, the writer would like to know how the wall movement of 0.001 of 4 in., or 0.004 in., was measured. Then, with the magical number, 0.42, as the initial value, and one other very doubtful point, Dr. Terzaghi constructs a smooth curve showing the relation between pressure and displacement, based on the results and figures noted previously. All cases must abide by this curve; if any test does not, it is in error by 37.5%, more or less, and yet the last paragraph of Dr. Terzaghi's discussion begins with the words: "The writer wishes to express his admiration for the remarkable precision with which Mr. Feld made his observations". The writer cannot see how he is expected to accept both statements at their face value.

The crux of the discussion is the discovery that Table 1 showed a horizontal component of $0.25 \frac{y h^2}{2}$ and a decrease of 3.4% with a wall movement of from $\frac{1}{18\,000}$ to $\frac{1}{4000}$ in., or 0.00634 cm.

Unfortunately for Dr. Terzaghi's thesis, the test proves just the opposite. As was stated by the writer in the paper, there is a remarkable lack of variation in the pressure with wall displacement, within the range tested. The writer submits the following notes and data taken from the log of the test while in operation:

"Attempt to obtain passive pressure and to measure the effect of wall movement on the active pressures.

"Zero readings of the scales (in order, for the location of Scales Nos. 1, 2, 3, 4 and 5). Scale-beam in lowest position: 108, 64, 60, 146, 125 (wall is tipped to left); fill at 5 ft. high, surface at 34° above horizontal, wood wall, vertical wall.

"Scale readings, beam in lowest position, 554, 464, 528, 985, 701. Release all scale stops (T-bars holding the scale-beams), beams move up $\frac{1}{2}$ in., vertical contacts move down $\frac{1}{200}$ in., horizontal contacts move out $\frac{1}{400}$ in.

"*Result.*—Slight sand movement, as shown by a little sand falling out at sides. Scale-beams just touching the trig loop at the top at the readings: 533, 451, 510, 951, 674½.

"Move riders of scale-beam till the scale-beam is in mean position; readings: 541, 459, 518, 965, 683. This corresponds to pushing the wall against the fill by $\frac{1}{800}$ in.

"Readings as the beam just touches the bottom of the trig loop: 553, 465, 527, 985, 701. This corresponds to another push of $\frac{1}{800}$ in. against the fill. Readings practically identical with the initial case. Effect of these movements is negative.

"Release all beams again and joggle each 3 times and repeat readings, as above.

"*Results.*—Vertical contact, none; horizontal contact, a few more grains fall out, movement actually occurs at wall.

Beam at top:	533, 450½, 510½, 950, 673.
Beam at bottom:	553, 465, 527, 981½, 700½.

"Release all beams again, joggle beams 3 times. Readings:

Beam at top:	532½, 450, 510, 950, 672½.
Beam at bottom:	552½, 464½, 526, 983, 700½.

"*Results.*—Negative. Pressures within movements tested ($\frac{1}{200}$ in.) are active, and the movement is not large enough to develop the passive pressure. Readings are not affected by the repetition of such movements. Variation is due to the different positions of the scale-beam. As the lower edge was the index in the lower position and the upper edge in the upper position, the total movement is $\frac{1}{2}$ in."

The difference in readings for the beam at the highest and at the lowest point depends on the type of scale, the leverage ratio, and the load. For the type of scales used, the difference in readings between the beam at the top and at the bottom varied from 8 to 30 lb., as the load varied from zero to 1 000 lb. The variation was quite uniform. A detailed description of the design and adjustment of the apparatus and the calibration of the scales is included in the writer's "Master's Thesis", copies of which are filed in the Library of the University of Cincinnati and in the Engineering Societies Library.

This test proves that, for the movements tested, $\frac{1}{16}\frac{1}{000}$ to $\frac{1}{200}$ in., there is no variation in the acting pressure and, therefore, it totally upsets Dr. Terzaghi's ideas.

Dr. Terzaghi is in error when he determines the maximum deflection of the wall as 0.07 cm., by assuming that the lowest horizontal timber is a simply supported beam. The wall acts as a plate or slab, with rib stiffeners in both directions, as well as cross-ties to decrease deflection. The maximum deflection occurs at some point near the center line at about the lower third

point of the height (somewhat above this point theoretically). Disregarding the effect of the cross-ties, the wall, considered as a slab, should have a maximum deflection of $\frac{1}{300}$ in. under the maximum loadings. If Dr. Terzaghi desires to find the deflection in the lower timber, the writer believes that he should consider it as a T-beam and include the effect of the sheathing. The different pieces were carefully fitted together and closely spiked.

From this maze of incorrect assumptions, Dr. Terzaghi derives the amazing conclusion that the results obtained are only 37.5% in error. If the equation, $3.4 \times \frac{0.07}{0.00634} = 37.5$, is considered, it must be remembered that the test described previously shows that the true value of the 3.4 is very close to zero. Hence, the error is not far from zero, according to Dr. Terzaghi's reasoning, as the denominator is a fixed quantity. The 0.07 is far too large, as the maximum deflection of the wall cannot be determined by the method used by Dr. Terzaghi. Even the 0.00634 cm. ($\frac{1}{1000}$ in.) is not correct, because the horizontal and vertical displacements were simultaneous. The results and the applications of the discussion are, therefore, worthless.

Dr. Terzaghi finds a remarkable agreement between the actual (0.15%) and his theoretical result (0.121%) for the variation of pressure with temperature. He is assuming that the effect is entirely on the fill and not on the wall. The temperatures are those of the air near the wall. The writer can hardly agree with the assumption, knowing that the effect of frost in this latitude seldom extends below 3 ft.

It seems to the writer that the numerical agreement has one other error—the signs of the two values are opposite. A drop in temperature causes a decrease in wall deflection. A decrease in wall deflection, according to Dr. Terzaghi's theory, causes an increase in pressure. The writer found a decrease.

As Dr. Terzaghi's entire discussion on the value of the concept of the internal friction and the internal resistance begins with the false premise: "Considering the fact that the large-scale Cincinnati observations confirm the existence of intimate relations between the intensity of the earth pressure and the distance of yield or deflection of the wall", there is no use in discussing the conclusions. A few points, however, are of interest. The method for the correct determination of the value of ϕ, according to Dr. Terzaghi, is to obtain it from the formula after the earth pressure has been found experimentally. The formula will then be checked; the writer agrees, but the final result is of no value; there is no way in which to correlate theory with experiment. The only method is to evaluate ϕ independently of the experimental determination of the pressure.

Dr. Terzaghi discovers that ϕ_0 for the material of the Cincinnati tests was rather less than 24 degrees. This angle is not the angle of repose, nor any angle of friction or resistance. It merely substantiates the statement that Dr. Terzaghi has no right to substitute the experimental value of the pressure in his formula to determine the value of ϕ. This ridiculous value comes from the use of the coefficient, 0.42. He finds the angle of internal friction to be far greater than the natural slope, a theory advanced by only a few experimenters, which has been disproved on theoretical grounds by

Boussinesq and Cain and on experimental grounds by practically every man who has shoveled earth or sand.

It may be of interest to some of the members of the Society to know that the ϕ_2 for loam and clay is given as a single value, but that, for sand, it varies by 17°, or a variation of almost 100% in the tangent. In his discussion of the idea of internal friction Dr. Terzaghi's reasoning is as follows: The pressure varies with the wall displacement, and if a constant mathematical relation exists between pressure and internal friction, the angle of internal friction varies with the wall displacement. The reasoning is logically sound; but logic guarantees not truth, but consistency with the value of the premises. The value of the premises has been shown previously.

The objections to the writer's method of determining the coefficients of friction and cohesion have some weight, from the scientific point of view. Dr. Terzaghi is arguing from the idea of soil mechanics and forgets that the paper is primarily an empirical solution of the problem. The methods used to determine the cofficients are an easy means for determining constants of the filling material for the purpose of a comparison between the experimental and theoretical values of the pressure. Two coefficients can be determined, as is explained in the paper. To facilitate comparison, it is assumed that they are the natural tangents of angles. That this assumption is not much in error is proved by the consistent results obtained in comparing the experimental with the theoretical values. The writer is free to admit that it is in error, theoretically, as the cohesion, if any cohesion is present, should be dependent on the load and, therefore, cannot be combined with the frictional resistance. Further research may cause a change in these ideas. The writer wishes to repeat his statement that, for an empirical method, the use of an angle the tangent of which is the coefficient of internal resistance, is advisable and quite accurate.

In connection with Dr. Terzaghi's distinction between two different types of frictional resistances, the writer submits the following ideas from Mayniel's book on earth pressure, printed in 1808.* Mayniel is generalizing the Coulomb theory to the case of sloped walls and states, as the bases of the problem, the following ideas:

"There are two types of friction in soils:

"1. Perfect friction, as in sand, is caused by the intercogging of the particles, acted upon by a continuous pressure, and is to be distinguished from cohesion which is a reunion of masses, like a gluing together.

"2. Imperfect friction is the rubbing or rolling of particles over each other, due entirely to their own individual weights."

The manner in which earth exerts pressure against a wall is:

"A certain part of the fill in the shape of a wedge with the apex at the heel of the wall slips down on a surface which is probably a plane; this material is not the entire volume which tends to slip, for later slips do occur if the wall slips or fails, but these do not concern the problem of the determination of the existing pressure."

Mayniel does not accept the assumption of the equality of the angles of friction and natural repose, although he mentions that Prony had made such

* "Traité Expérimental et Théoretique sur la Poussee des Terres", 1808, Paris.

an assumption, but makes no attempt to evaluate the internal friction. The mathematical work is very detailed and complicated, but he states: "That is because of the generality of the solution of the problem; we must take into account all the points of the problem." Therefore, it seems that a cycle has been completed, and the subject, from the theoretical point of view, is back to the solution of Mayniel.

Dr. Terzaghi misinterprets the writer's ideas concerning cohesion, so that he submits an elaboration of the few statements made in the first part of the paper. In the interior of a mass of earth in the usual state, the grains being of irregular shape and of non-uniform size, and coated with water films as well as with colloidal substances, equilibrium of forces must exist, for there is no displacement. The water film is practically never absent, although in dry soils it may not completely fill the air spaces between the particles. The colloid matter consists of such substances as ferric and aluminium hydroxides, humic acids, and decomposition products of living plasms. The surface tension in these films tends to draw the particles into closer contact—to bring the centers of gravity of adjacent particles closer, that is, to decrease the voids and increase the unit weight. In doing so, it tends to squeeze out some of the liquid between particles, and equilibrium results at a definite point. This explains why the unit weight of a soil is a maximum at a definite moisture content and is smaller for less and greater moisture contents. Excess water refuses to permit the particles coming closer together; too little water means that there will be some air in the capillary spaces between grains. A mass in internal equilibrium, therefore, is at its maximum unit weight. The mass can only be compacted into a smaller volume by a load sufficiently large to crush the particles, that is, to shear off the rough corners and thin edges.

The conditions in the writer's method of determining the coefficients do not differ greatly from those which exist at the state of equilibrium, just before the force is exerted on the bottomless box. A pull on the box tends to cause a separation along the weakest surface, which is a rough surface, averaging very closely to the plane of the bottom edges of the box. As the grains do not have horizontal faces along this plane, horizontal separation can only occur after a slight vertical displacement, and this, as in the case of friction between solids, is a sliding along the sloped faces in contact. The resistance to this displacement, therefore, is the sum of the following resistances: The force of gravity on the upper mass, the friction between the grains in contact, and the surface tension of the liquid.

As soon as vertical separation has been effected, with a slight simultaneous horizontal displacement, the particles along the surface of separation are no longer in such close contact, the film is broken at several places, and the force required to keep up motion, which may be compared to the skimming of a rough object over rough ice, that hits only the high spots with an occasional shearing off of a peak or rotation of a loose grain to clear the path, is smaller than the initial force.

The coefficient of internal resistance, therefore, is a measure of the sum of the following main resistances:

(1) Gravity tending to prevent upward motion.
(2) Surface tension of the liquid materials between particles. This is a very large quantity in colloidal soils.
(3) Friction between grains which have a relative displacement. This is decreased by the lubricating action of colloids, but is independent of the motion if the lubrication is disregarded.
(4) Elastic compression of the particles, for the total load is now carried by a relatively few point contacts and the unit stress is proportionately higher.
(5) Rotational resistances of the particles opposing the motion, bringing into play more surface tension, friction, and compression.

As soon as motion is started, the resistance drops to the value called the coefficient of internal friction times the load. Three of the five factors enumerated are decreased, whereas the third and the fourth may increase. As the mass moves forward, it bobs up and down as the grains separate and intercog, but the amount of rise and fall of the center of gravity of the moving mass is less in magnitude than the original rise. The surface tension still acts, but the area in contact is now less and the thickness of the film is greater, so that the resistance is decreased for two reasons. The total frictional resistance remains the same and may increase somewhat because the lubricating action will decrease with the smaller areas of contact and the larger voids into which the liquid can be squeezed. The mass now being in a higher position, fewer grains oppose the motion, and the resistance offered by this factor is decreased.

In dry sand, there is no surface tension and no lubricating action, the two factors which give the largest part of the difference between resistance at rest and resistance during uniform motion. It should be expected to find the two coefficients for dry sand of approximately the same value. This is substantiated by experiment.

Dr. Terzaghi has shown that a re-arrangement of particles takes place in a soil under pressure even when lateral displacement is prevented. In this way, he discredits his determination of the modulus of elasticity by a similar method. As load is placed on a retained soil, part of the burden is slowly shifted to the container, if the latter is sufficiently stiff to prevent part of the lateral displacement of the grains. The maximum amount of this relieved load is dependent on the friction between the soil and the container. Dr. Terzaghi will have to take account of this, as well as of the effect of shape on the load dissipated, before his mathematical analysis will be even of approximate value. The various symbols used are not defined, so that the real meaning and application of the formulas derived is not determinate.

As may have been noticed from the comments just made and also from Dr. Terzaghi's conclusions, the disagreement which he finds is caused chiefly by his refusal to accept the definitions. Under such conditions, constructive criticism and discussion is impossible.

The writer still believes that the ideas advanced by Dr. Terzaghi are valuable only from the qualitative point of view, and that his quantitative rela-

tions are in great error. The numerical values given in the theory have no real experimental basis and cannot be used in the problem of the paper, the accurate determination of the lateral pressure of earth behind a retaining wall.

Mr. Goldmark deserves the thanks of the Profession for placing on record a set of field tests which were sufficiently comprehensive to yield results that could be used for the design of such a large project. The important fact is that these tests were made on the job and not in a laboratory, with the material as it existed in the ground. This report shows that the much needed experimental study of the soil is possible and will not require expensive apparatus or excessive time to change the assumptions on which the design is based from a pure guess to an experimental basis.

The material tested probably contained a large proportion of colloidal matter, which explains the large variation in unit weight with moisture changes, as colloids have a high absorption capacity, the action during drying, resulting in hard bricks with large cracks and a porous structure, the caking in the small box, coagulation along the walls forming a water-tight container for the inner mass which remained liquid for a long time, and the variation and low values found for the angle of repose.

In calibrating the steel plate, it might have been advisable to approximate the actual conditions of pressure against the plate a little closer. Instead of determining the modulus of elasticity from the deflection of the plate supported as a simple beam with a concentrated load in the center, it might have been loaded by a distributed load from zero at one end to a maximum at the other, the latter to be used for the bottom of the test wall. The maximum deflection will occur at $L\sqrt{1-\sqrt{\frac{8}{15}}}$, or 3.23 ft. from the support at the zero load, corresponding to the top of the wall. This is very nicely checked by the curve on Fig. 24, "Bending due to water", with a maximum between 3.15 and 3.20 ft. The writer hopes to hear of other field tests of a similar nature and value in the near future. It will be helpful for the comparison of results and the application to other locations, if a complete physical and chemical analysis of the material is made.

Mr. Fargher's questions have already been answered, in more or less detail, and, to save space, the writer refers him to the previous part of this discussion. It will be very interesting to see the results of Mr. Fargher's experiments with a wall made up of vertical parallel strips, a device which has not been tried heretofore. The writer hopes that the pressures on each strip will be measured independently, and that some other material than "artificially dried sand" will be used.

As was noted carefully in the paper, the remarks on Mr. Crosthwaite's experimental work were based on his 1916 paper and the abstracts of his 1920 paper. At the time the writer's paper was prepared, a complete copy of the 1920 paper could not be obtained. Since then, the writer has read the complete paper, and finds that there is no necessity for revising any statement made by him. The references and quotations prove the statements made. The writer can see no justification for calling three different numerical characteristics of a

soil by the same name. With the exception of Mr. Crosthwaite's, the writer has not found a single reference to any method for the determination of the angle or coefficient of internal friction not based on the idea of internal shear. Measuring the angle of internal friction by the use of a plunger in a bucketful of soil is open to numerous indeterminate and undeterminable errors, as is well proved by the variation in the values found. As the writer has already stated, the angle of internal friction cannot be determined by measuring the pressure against a wall, unless the accuracy of the theoretical formula is assumed; and, as Mr. Crosthwaite points out, there is no known correct formula or theory.

In discussing the value of the method of measuring the pressures in the writer's apparatus, Mr. Crosthwaite claims that two errors exist: That due to the buckling of the test wall and the resultant "kick-back" on the horizontal components, and that due to the lost vertical component caused by the relative motion between the wall and the fill. In answer to the first point, the writer refers Mr. Crosthwaite to the description of the apparatus and to several remarks in the previous part of this discussion. He will find that the lug angles attached to the wall prevented any motion against the fill. The wall, therefore, was in a fixed plane adjacent to the plane of the filling material. Any deformations which occurred locally were certainly not much larger than, if as large as, the elastic deformations in an actual retaining wall.

The discussion relative to the second point makes the writer believe that Mr. Crosthwaite is suggesting the existence of two vertical forces, one of which is part of the earth pressure, *per se*, and the other is the vertical frictional force caused by the horizontal pressure. Mr. Crosthwaite claims that the writer's apparatus only measures the former, and he agrees that the thrust (in the paper the writer used the expression, "the horizontal thrust") is independent of the nature of the wall. How, then, can Mr. Crosthwaite explain these two facts:

1.—That the ratio of the vertical to the horizontal component was in all cases equal to the coefficient of wall friction, which coefficient was determined independently.

2.—That the vertical components in walls of the three types—wood, glass, and sheet-metal—were not the same; but, in each case, the values were as previously stated.

Mr. Crosthwaite's objections might be permissible if the movements of the wall were of appreciable magnitude, but, as it is, they were smaller than the total elastic deformations in the several grains making up the fill, to say nothing of the void spaces between the grains. There was no relative motion between the wall and the fill. The force measured was the maximum possible before such motion could occur, as has been explained before. For a detailed analytical and mathematical analysis of the measuring devices, showing the necessity and sufficiency of five points of support, the writer refers the reader to Mueller-Breslau's book.

The writer wishes to express his thanks to those who aided him in the preparation of the paper and of his discussion, and also to those who discussed the paper, giving of their time and knowledge to enrich the contents by new

ideas and methods of attack. He was most pleasantly surprised to receive discussions from foreign countries, and regrets that lack of time prohibits a more thorough exchange of views before the publication of the paper. He hopes that those who have discussed the paper, and any others who may be interested, will feel free to continue the discussion, by mail or otherwise, should any statement require further elaboration. He hopes that he will be able to present another paper on the same subject in a few years containing more decisive experimental work and perhaps a simple and consistent theory for the accurate determination of the lateral pressure of earth.

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ADDRESS AT THE ANNUAL CONVENTION IN CHICAGO, ILLINOIS, JULY 11, 1923

By Charles F. Loweth,* President, Am. Soc. C. E.

There is a Constitutional provision that the President of the Society shall present an address at each Annual Convention. After consideration of how this requirement could best be complied with on this occasion, it seemed quite opportune to review some aspects of the affairs of the Society, both in retrospect and prospect. To do this will obviously result in repeating some matters which may be well or even better known to many; to look forward may provoke renewed interest in the Society's aims and activities. It is hoped that what follows may not appear to be a needless repetition of facts, now more or less of record, but may help to a better understanding of the problems with which the Society is confronted and be helpful in their wise solution.

The American Society of Civil Engineers was founded November 5, 1852. It has therefore just passed its Seventieth Anniversary. In these seventy years, much water has gone over the dam of engineering practice; much of the whole field of engineering has been revolutionized in this period, both as to materials of construction and as to practice. It is safe to assert that the Society and the rank and file of its membership have contributed in large measure to, and have had a great degree of responsibility for, the remarkable progress of these seventy years.

It is interesting to note the date of the founding of National engineering and architectural societies in various countries:

The Institution of Civil Engineers of Great Britain was founded in 1818 and incorporated in 1828.

The Institution of Civil Engineers of Ireland was founded in 1835 and incorporated in 1877.

The Royal Institute of British Architects was founded in 1837.

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The Institution of Mechanical Engineers was founded in 1847.
Koninklijk Instituut van Ingenieurs, (Holland) was founded in 1847.
Oesterreichischer Ingenieur- und Architekten-Verein (Austria) was founded in 1848.
Société des Ingénieurs Civils de France was founded in 1848.
The American Society of Civil Engineers was founded in 1852.
The American Institute of Architects was founded in 1857.
The Iron and Steel Institute was founded in 1869.
The American Institute of Mining and Metallurgical Engineers was founded in 1871, the organization meetings of this Society having been held in the rooms of the American Society of Civil Engineers.
The American Society of Mechanical Engineers was founded in 1880.
The American Water Works Association was founded in 1881.
The American Institute of Electrical Engineers was founded in 1884.

The older local engineering organizations of the United States are the Franklin Institute, founded in 1824, the Boston Society of Civil Engineers, in 1848, and the Civil Engineers Club of the Northwest, now the Western Society of Engineers, founded in 1869.

It will be interesting to review the early efforts to found a National engineering society in this country. The founding of this Society in 1852 was preceded by earlier efforts; more than twelve years previously "a highly respectable meeting of the profession" was held in Augusta, Ga., of which, unfortunately, no record remains. At this meeting, the formation of a National engineering society was urged, and an appeal to that end was sent out to a widely scattered group of men, calling a convention of civil engineers of the country. In response to this call, forty engineers, from nine of the Seaboard and Gulf States, and from Illinois and Missouri, met in Baltimore, Md., on February 11, 1839. At this meeting, it was decided to organize a Society of Civil Engineers of the United States. Mr. Benjamin H. Latrobe, of Baltimore, a bridge engineer of great note, was elected President and Mr. J. F. Houston, of Pennsylvania, was made Secretary. A representative committee of seventeen, from eleven States, so elected that "different portions of the Union may be represented so far as is practicable," was appointed to prepare a Constitution and take other steps toward forming a permanent society. Two of these gentlemen were later members of this Society. A few members of this committee met in April, 1839, at the rooms of the Franklin Institute in Philadelphia, Pa. A draft of a constitution for a society was agreed on, and later submitted to the remaining members, but failed of approval, as only seven of the seventeen members voted in favor of it. It is worthy of note that the name of the proposed society, as stated in the draft of the Constitution submitted by this committee, was identical with the present name of the Society, and that architects were to be admitted only as Associates. This effort, however, seems to have been born out of time, for nothing further resulted; however, it doubtless paved the way for the more successful effort thirteen years later, in November, 1852, when this Society was organized.

The founding of this Society was the result of a call by a few New York engineers, dated October 23, 1852, for a meeting to be held November 5, in New York, N. Y., to organize a "Society of Civil Engineers and Architects." At this meeting, it was resolved that an organization should be formed and incorporated, the name of which should be "The American Society of Civil Engineers and Architects." A draft of a Constitution was submitted and approved, and remained in force without change until 1868. It fixed the permanent place for the transaction of the business of the Society as New York, N. Y., and provided for one grade of membership only, to which were eligible "Civil, Geological, Mining and Mechanical Engineers, Architects and other persons who by profession, are interested in the advancement of science." Officers were elected, the first President being the late James Laurie, who remained in office until after the re-organization of the Society, in 1867. At this meeting was approved the draft of an address to be sent to those engineers and architects of the United States, who were considered by the Board of Direction as eligible for membership, making an appeal for support for the new society. A part of this appeal is as follows:

"It will be admitted that no point in our country offers the facilities for rendering such a society of practical benefit to the public as well as to its own members as the city of New York, and so long as this city retains its present commercial importance, so long it will be a center around and within which there will accumulate by a natural law practical commercial and professional information not elsewhere to be sought, and which, embodying the elements of successful enterprise, may be regarded as a fund of valuable data, equally valuable to the man of science and to the political economist. Much of this information, is, from its nature, unwritten, but is entirely accessible, and, under the auspices of a society formed for that purpose, could be rendered available to such members as may desire the benefit of it.

"The Constitution of the Society declares that it has for its object:

"The professional improvement of its members, the encouragement of social intercourse among men of practical science, the advancement of engineering in its several branches, and of architecture, and the establishment of a central point of reference and union for its members.

"Among the means to be employed for attaining these ends shall be periodical meetings for the reading of professional papers, and the discussion of scientific subjects, the foundation of a library, the collection of maps, drawings and models, and the publication of such parts of the proceedings as may be deemed expedient.

"Civil, geological, mining and mechanical engineers, architects and other persons who, by profession, are interested in the advancement of science, shall be eligible as members.

"It is anticipated that the union of the three branches of civil and mechanical engineering and architecture will be attended by the happiest results, not with a view to the fusion of the three professions in one; but as in our country, from necessity, a member of one profession is liable at times to be called upon to practice to a greater or less extent in the others, and as the line between them cannot be drawn with precision, it behooves each, if possible, to be grounded in the practice of the others; and the bond of union established by membership in the same Society, seeking the same end, and by the same means, will, it is hoped, do much to quiet the unworthy jealousies which have tended to diminish the usefulness of distinct societies formed heretofore by the several professions for their individual benefit.

* * * * * *

"It is scarcely necessary in this place, even did space permit, to enlarge on the many advantages which may be anticipated to flow from the judicious management of such a society as the one we have organized."

The first meeting at which the Society actually got to work as such appears to have been January 5, 1853. However, only eight members were present at that meeting.

Even in 1852, the time was premature for the formation of a National civil engineering society; the early records state that, although in 1853 there were eight regular meetings and a membership of fifty-five, the average attendance at each meeting was only six, and, in 1854, with six meetings and no increase of membership, the average attendance per meeting was less than six. This was a discouraging situation and, with the hope of bettering it, the dues were reduced from $10 to $5 for resident members and from $5 to $3 for non-resident. The report of the Board of Direction states that it was induced to make the reduction "in order to render membership less onerous, with the hope that new members might be induced to seek a connection with the Society," and, further, that "the labors of the members more than their money is wanted to make the Society useful." There appear to have followed only two or three meetings in the early part of 1855. During these two years and a few months, the Society had no rooms it could call its own, which fact was thought at the time to be in some measure responsible for its lack of growth.

Then follows a lapse of about twelve and one-half years, during which the Society, although it did not disband, appears to have hibernated. At the call of the President, Mr. James Laurie, who still retained office, a meeting was called for October 2, 1867 to "take such steps as may be necessary to resuscitate the Society." At this meeting a committee was appointed to prepare a "plan for the revival of the meetings." At a meeting held a week later, this Committee reported and, among other things, recommended the acceptance of an offer for the use of certain rooms at an annual rental of $400, and their furnishing at a cost of $600. The next meeting, a month later, on November 6, 1867, was the first held by the Society in rooms devoted exclusively to its use. At this meeting the accumulated dues of members during the previous twelve or more years were remitted, and the annual dues were increased to $10 for resident and $5 for non-resident members. During this intermission in the activities of the Society, its financial affairs seem to have prospered. In 1854, the Treasurer had $266.93 on deposit in a savings bank which, on the re-organization of the Society, had grown to $497.57, and the President turned over $558.25, representing the dividends on five shares of New York Central Railroad stock which the Society had acquired in March, 1855. At the following meeting, December 4, 1867, while only nine members were present, fifty-four new members were elected. This meeting was the beginning of the custom of holding regular semi-monthly meetings on the first and third Wednesdays of each month, a practice which prevailed until the formation of the New York Section in 1921, since which time the Society meetings have been held monthly. On March 4, 1867, by a vote of seventeen

to four, the name of the Society was changed to the "American Society of Civil Engineers." Through oversight, incorporation was not effected until April 17, 1877.

The record shows that in the premature efforts made in 1839, and throughout the years 1853-54-55, and again when the infant Society was being revived in 1867, there were uppermost in the minds of the men working for the formation of this Society two outstanding principles: First, that it should be broadly inclusive of all Civil Engineers, including the allied profession of Architecture; and, second, that it should be geographically inclusive of all the United States. At no time does it appear that the founders would have been satisfied with a merely local Society. Is it not quite possible that the quarter of a century of more or less continuous effort on the part of the little handful of pioneers who labored to found this Society might have borne fruit much more quickly if their vision had been less far-reaching? But, if so, it is more than probable that the National Engineering Societies, of which the American Society of Civil Engineers is the oldest, would not have the large membership, influence, and prestige which is theirs to-day.

A diagram showing the growth of membership of the four Founder Societies from their foundation to January 1, 1923, discloses that the growth of the Society from its rebirth in 1867, for twenty years, although slow, had a constant increase. For the following period, to about 1905, the rate of growth was more rapid, and from that date until within the last few years, it has been at a still greater rate and has compared favorably with that of the other Founder Societies. Quite recently the rate of growth in membership seems to have slackened noticeably. This fact may be accounted for largely by the lack of employment, unsettled conditions, and unrest which have prevailed so generally following the World War. The increase in membership of late appears to be more nearly the normal rate of earlier years, and it is confidently anticipated that the growth of the Society will continue to be at least as satisfactory as it has ever been. There are a large number of professional engineers who are eligible for membership, who, if they availed themselves of the privileges, would, through the activities of the Society, advance the profession and benefit themselves. A high standard for membership qualifications has ever been maintained; this is of the first importance and must in no wise be sacrificed for increase in numbers. The Society's service to the Engineering Profession, and as a means of service by the profession to the country, are so well established that there can be no question of its permanency. Clearly, what is needed at this time is a continuation of those qualities which hitherto have contributed so largely to the prosperity and usefulness of the Society, namely, an abiding interest and loyalty on the part of the members, an adherence to those policies which have been found by past experience to make for progress and usefulness, and such sane progressiveness and vision as will meet the needs of our day and generation.

Annual Conventions

It has been the practice of the Society to hold an Annual Convention each year during the summer months. This practice was begun in 1869. The notice announcing the time and place of the first Convention, which was held in New York, N. Y., was dated May 10, 1869, and read, in part, as follows:

"Our Society has of late been impressed with the importance of annually gathering its members in Convention for the interchange of ideas and the discussion of professional subjects, as well as those relating to the extension of its own field of usefulness.

"These objects cannot be attained by means of the twenty-three regular meetings, held during the year, on account of the distant residence of many of its members, but they require the appointment of some day on which all can meet and strengthen that bond of sympathy which should animate hearts engaged in the furtherance of a common cause.

"It is proposed, therefore, to hold the first of such a series of conventions on the 16th day of June, 1869, commencing at 10 A. M.

"Through the courtesy of the Chamber of Commerce, the meeting will be held in their Assembly Room. It will occupy most of the day, and be followed by a suitable collation.

"The Society relies on the individual efforts of its members for sustaining the interest of the occasion, and desires that you will either come prepared to read a brief professional paper—in case of unavoidable absence will forward such to the Secretary—or else be prepared to make some statement of engineering experience calculated to prove of general interest.

"An early and definite reply on your part is earnestly requested."

In response to this call for the First Annual Convention fifty-five members attended and five formal papers were presented. Of the Second Convention, held the following year, also in New York, N. Y., the report states that the "proceedings were interesting, the spirit of the members excellent, both socially and professionally, but the number present was less than anticipated and hoped for." "This," states Charles Warren Hunt, Secretary Emeritus* of the Society, in his very interesting, but all too brief, "Historical Sketch of the American Society of Civil Engineers," published in 1897, from which much of the historical data contained herein have been taken, "led to the movement to hold future conventions in other localities of the Union, with a view to strengthening the national character of the Society." The Third Convention, in 1871, was again held in New York, N. Y., and the fourth, in June, 1872, was held in the City Council Chamber in Chicago, Ill. The "Historical Sketch", referred to previously, states that it was well attended, twelve papers were presented, and a dinner was served at the Parker House. The Fifth Convention was held in May, 1873, in Louisville, Ky., and the Annual Report refers to it as having been attended by the largest number of civil engineers ever assembled together on this continent; the attendance, from an unofficial source, is given as 79 members.

For some reasons not clear, the Sixth Annual Convention, that of 1874, was again held in New York, N. Y., but was the last held in that city. The first Convention held west of the Mississippi River was at St. Louis,

* Secretary of the Society from 1895 to 1920.

Mo., in 1880, and the Convention of the following year, the first outside the United States, was held in Montreal, Que., Canada. In 1886, the Convention reached the Rocky Mountains at Denver, Colo., and the Pacific Coast at San Francisco, Calif., in 1896; the following year, it was held at Quebec, Que., Canada. Only three times since then have Conventions been held outside the United States: In London, England, in 1900, in the City of Mexico, in 1907, and in Ottawa, Ont., Canada, in 1913. There have been no interruptions of these Annual Conventions since their beginning, except in 1917 and 1918, when none was held on account of the World War. Formerly, the places for holding the Conventions were selected somewhat at haphazard, but lately they have been determined by an orderly rotation by Districts.

At various times there has been a demand for more gatherings similar to the Annual Conventions. As long ago as 1879, the late Octave Chanute, M. Am. Soc. C. E., then a Director and, later, President of the Society, proposed the following:

"*Resolved*, that it is the sense of this meeting that the Board of Direction should take immediate steps to provide for holding in various cities, of three general meetings in each year, in addition to the general convention, for purposes of professional intercourse."

This motion was lost. Later, in 1913, in addition to the Annual Convention held in June of that year in Ottawa, Ont., Canada, a general meeting was held in October in New Orleans, La., at which the attendance nearly equalled that of the Convention. Nothing more was done in this matter until 1922, when general meetings were held in April at Dayton, Ohio, and in October at San Francisco, Calif., both of which were as well or better attended than the Annual Convention held at Portsmouth, N. H. This year, in addition to this Convention, a general meeting was held in April in New Orleans, La., and one will be held in October in Richmond and Norfolk, Va. The Board of Direction has decided to continue this policy for another year and plans, as yet tentatively, to hold a general meeting in the spring at some place south of the Ohio River, and in the fall in Detroit, Mich., these meetings to be in addition to the Annual Convention which will be held in June in Pasadena, Calif. Whether these general meetings, held away from Society Headquarters, will become a permanent policy will depend on the interest taken in them by the membership. It would seem that with such a large and widely scattered membership two general meetings and the Annual Convention could very profitably be held annually; they should serve to stimulate the interest of the members in promoting the welfare of the Society, afford greater opportunity for participation in its activities, and help make it more truly a National society. The main purposes of the Convention and general meetings, and as well the Annual Meeting in January, held in New York, N. Y., cannot be better stated now than was done by President Julius Walker Adams at the Annual Convention of 1874, when he said:

"The chief purpose and end of our convention, unlike most gatherings known by this name, is to promote social union among the members of this Society * * * [those absent] will lose the hand-shaking, the per-

sonal interchange of the sentiment of fellowship and brotherhood, the knowledge of each other, that basis of true *esprit de corps,* which is the life and soul of a Society like this."

The attendance at, and interest in, the Annual Conventions has not been maintained in keeping with the growth of membership. The Board of Direction and the local committees in charge of planning these features of the Society's activities devote considerable time and thought to them and they are worthy of a more loyal attendance by the members.

Local Sections

The problems confronting a National society are in many respects different from, and at the same time more involved than, those with which local societies have to contend. Although the prestige of a National society attracts many, the higher dues and qualifications for membership offset this advantage. Only a comparatively small proportion of the members of a National society can be in touch with its activities, and then only at infrequent intervals, whereas the opposite is true with respect to local societies. These handicaps were early recognized, and much thought has been given at various times to devising means to make membership in the National societies not only attractive, but increasingly useful. One such means has been the provision for Local Sections.

The problem of Local Sections is an old one. As far back as 1885, a provision for Local Sections was favorably considered by the Board of Direction in a resolution offered by no less an authority than the late A. M. Wellington, M. Am. Soc. C. E. Repeatedly, thereafter, the Board of Direction saw the need of some means through which the interests of the ever-widening geographical distribution of the membership could be placed in closer touch with the Society's affairs, more effectively aid in its upbuilding, and make it of more value.

In 1905, the Board directed that inquiries be sent out to a large number of members scattered throughout the country to ascertain the feeling with respect to the establishment of Local Sections. At that time, there were only two Local Sections, one at Kansas City, Mo., and the other at San Francisco, Calif. The result of this canvass was far from unanimous in favor of Local Sections, and from some localities there was a decided expression of disapproval. It is interesting to note that the strongest objections came from some cities which now have the strongest Local Sections. The objections were perhaps natural in those cities in which local engineering or technical societies were already functioning, and especially where they included in their membership a larger number of engineers than the local membership of this Society. There is enthusiasm in numbers and more engineers would rally about a large and successful local society than a Local Section of a National society with fewer members. However, with the growth of other National societies came a spirit of rivalry and intensified interest, which, together with the natural increase in local membership and the awakening desire on the part of the members to put more service into and get more

benefit out of their Society, brought about a change with respect to Local Sections, making them increasingly popular until, at the present time, there are forty-one, all more or less perfectly functioning. However, the problem of Local Sections is one of complexity and is yet unsolved. The Board is desirous of finding some way in which the Sections can function more effectively and with more co-ordination with the governing body of the Parent Society, so as to make them, in a larger sense, the local representative of the Society and a means of stimulating local interest in it and in all that it represents. In the last analysis, this much seems sure—there must be a recognition of a mutuality of interest, helpfulness, and confidence between the Sections and the Parent Society; unless there is loyalty, they both fail. This is a National Society, it has cost much to make it such; to keep it such, it will be necessary for the Sections to take a broad view of the Society's aims and usefulness and their relation thereto and to guard against an undue emphasis of sectional issues.

It is realized that what has been said here does not answer the question often asked: "What are the purposes and possibilities of Local Sections?" Naturally, the answer will depend largely on local conditions; but is it not fair to assume that generally the purposes and possibilities of the Local Sections of this Society may well be similar to those of a local engineering society, and have, in addition, through the bonds of a strong National society, the advantages of a close association and co-operation with a nation-wide group of men of like mind and aspirations?

Meetings of Board of Direction

The growth of the Society and its nation-wide character have been largely promoted by three important policies adopted in recent years. These policies are: The allowance of mileage to members of the Board of Direction for attendance at Board meetings; the holding of less frequent Board meetings; and by holding Board meetings at intervals away from Society Headquarters.

Payment of mileage for attendance at Board meetings was begun in 1908 and resulted immediately in a larger attendance. Previously, there were repeated instances in which Directors did not attend any of the Board meetings during the entire two or three-year terms of their office. This condition of affairs applied not only to the representatives of districts remote from New York, but was true in lesser degree for many districts not far from Society Headquarters. The mileage allowed at first did not fully cover the necessary expenses incurred in attendance at Board meetings, and it was later increased sufficiently to do so.

Payment of mileage, however, was not of itself sufficient to insure attendance of officers at Board meetings. Prior to 1916, there were ten or more regular Board meetings a year, each meeting generally lasting only one day. Not many officers were able to spare the time for attendance at so many meetings, especially if more than a few hours' travel was necessary to and from Society Headquarters. In 1915, regular Board meetings were reduced to five or six, lasting generally two days; more recently, a change has been

made whereby meetings are held quarterly. This materially reduced the demands on the time of officers and resulted in an increased attendance.

Formerly, all Board meetings were held at the Society's Headquarters in New York, N. Y., except one each year at the Annual Convention. Lately, the Board has met two or more times each year at points other than Society Headquarters. In 1922, Board meetings were held at Dayton, Ohio, Portsmouth, N. H., and San Francisco, Calif. This year, meetings have been held at New Orleans, La., and in this city (Chicago), and one will be held in October in Richmond, Va. All these meetings have been held immediately preceding the Conventions or general meetings of the Society held at the same places. Board meetings held at widely scattered points and at intervals make it easier for members of the Board to be active, stimulate the interest, not only of the officers, but of the membership in the various districts where such meetings are held, and places the Board in closer touch with the needs and activities of the Society.

Reference to the record of poor attendance at Board meetings during early years should not be interpreted, at least for the most part, as a lack of interest on the part of the officers, but to practical difficulties. Comparatively few men had such command of their time and were financially able to incur the expense of attending ten meetings a year, eight or nine of which were held in New York. Under such conditions, quite naturally, the resident officers dominated the affairs of the Society, as, under similar conditions, would the members in Chicago, Ill., or San Francisco, Calif., Washington, D. C., or Omaha, Nebr. My observation has been, and it is, I think, attested by the record of the Society's growth and prosperity, that during these earlier years, in which the burden of directing the Society's activities were so largely borne by the resident officers, that they served the Society unselfishly, loyally, and efficiently. The Society owes a debt of gratitude to that devoted body of resident members who, through so many years, were mainly responsible for its management.

Technical Divisions

The number of engineering societies is legion. It is difficult to state the exact number as they merge by many gradations from those organized more or less for social purposes, through the various lines of technical activities, to those having high professional qualifications for membership. It has sometimes been said that if this, the oldest Engineering Society, had had a proper vision it would have become the one all-inclusive engineering society of this country. It may well be doubted whether this would have been possible, but, if so, it is more doubtful whether the outcome would have been desirable. The multiplicity of engineering societies has been of great value to the profession in stimulating professional activity and solidarity. Their competition, although expensive, has been helpful and inspiring, and certainly no one organization, or even a few, could have accomplished the work which has resulted through the efforts of the many. Especially is this true in view of the fact that many of the smaller societies, such as the American Railway Engineering Association and the American Society for Testing

Materials, are so highly specialized that they cover their particular fields of activity probably much more efficiently than could be done by a large general society. Be this as it may, it would seem certain that the time has now come when there should be little or no need for founding new societies. The recognition now given to Local Sections by the several National societies will tend to check the formation of new societies, and the same end will be effected by the policy recently adopted by the Board of Direction in the provision for Technical Divisions.

Technical Divisions may be formed by twenty or more members of the Society who may be especially interested in some branch of engineering. They are given a large degree of autonomy in the charge of their affairs, may hold their own meetings in charge of their own officers, and conduct their own committee work. There are now organized and more or less perfectly functioning, a Highway Engineering Division, an Irrigation Engineering Division, a Power Division and a Sanitary Engineering Division. Other Divisions will doubtless be formed in the near future. A canvass of the membership was recently taken for the purpose of ascertaining the wishes of the members as to additional Divisions. The responses, although not as numerous as expected, were encouraging. A Division of City Planning would seem desirable and is now being organized. City Planning has recently come to the fore as a comparatively new and somewhat neglected field for civil engineers. In this connection, it is interesting to be reminded that, at the first regular meeting of this Society, on January 5, 1853, the topic of the evening was "The Relief of Broadway," and Mr. James Laurie, the first President, presented a plan for the placing of railway tracks above the level of the street. This matter was further discussed at the next meeting, as was also "Other Projects, Including the Widening and Extending of Church and Mercer Streets." With a record like this, the members of this Society should not permit present-day problems of City Planning to pass into the hands of another profession, where it would appear they do not as properly belong.

The formation of these Technical Divisions makes it possible for small groups of members interested in particular subjects to get together more easily and function more efficiently than would be possible by the foundation of a new society intended to cover only a restricted field.

Co-operation

Much has been said of late of co-operation of Engineering Societies as if it was something new. It is not new. There has always been a goodly degree of co-operation; however, the great increase in the number of such societies and the fact that many of them are so highly specialized has seemed to call for an increased co-operation and, at the same time, made it increasingly difficult to bring about.

The membership requirements of some societies stress most strongly technical qualifications, in fact, in some the technical qualifications are highly specialized and are fully met by manufacturers and expert technicians. In others, the outstanding qualifications for membership are those of professional attainments and, in some of these, professional attainments of a highly

specialized nature. In some, social or local affiliations are a large factor in membership qualifications. It must be obvious, therefore, that under such widely varying society ideals, co-operation will necessarily be restricted to those matters of common interest. These may be local or National affairs, may be purely technical matters, or be those of a professional character; in each case, the incentive for co-operation will vary. The incentive and opportunity for co-operation depends, therefore, in a very large degree, on a common interest, the purposes and ideals of the several groups and equally so on an approximate equality of ability for the furtherance of that common interest. With such a multiplicity of Engineering Societies as now exist, co-operative efforts, if unduly forced, will naturally tend to a compromise of essentials or to the over-emphasis of many things of minor importance. In such case, the danger would be to the societies of greatest prestige and highest ideals.

This Society has recently, and for the third time, declined to join with certain others in an all-inclusive plan of engineering society co-operative work. There is ample evidence that these decisions have been based on the plan of, and not on the spirit of, co-operation. Clearly this is so, for the whole history of the Society shows, time and time again, a deeply rooted disposition to co-operate fully and freely whenever there has been opportunity to do so effectively. At the present time, there is a very large degree of co-operation between engineering organizations in technical matters. The extensive and excellent work being done by the Engineering Standards Committee, which is sponsored by a number of the engineering societies and participated in by many of their members, is a co-operative effort of high order; the same is equally true of the excellent work of the Division of Engineering of the National Research Council. Much other technical activity is being fostered through the joint agencies of various engineering societies. It makes little difference whether this is accomplished through temporary agencies, such as joint committees, or more permanent agencies. The American Society for Testing Materials and the American Railway Engineering Association may be considered, in view of the fact that many of their members are also members of other less specialized societies, as co-operative organizations by engineers. Effective co-operation calls for united effort and, equally so at other times, that one party shall refrain from duplicating the activities of the other. The setting up of Engineering Foundation for the furtherance of research in engineering and of United Engineering Societies for the management of the joint property interests of the four large National societies, called by these acts the Founder Societies, are other notable instances of co-operation. In other than technical matters, co-operation has not progressed as far, although it is far from being negligible.

In those matters pertaining to the service of the Engineering Profession to the country and those pertaining to the general welfare of the profession, there is room at this time for more concerted activity on the part of engineering organizations. Considering these in their broadest aspects, it would seem as if it was the especial obligation, and as well the privilege, of the large National societies, particularly those in which the professional qualifications

for membership are the highest, and which for these reasons have the prestige and wealth of personnel, leadership, and funds beyond those of local or specialized societies, to undertake primarily and largely the burden of representing the Engineering Profession at large. In that case, therefore, the Founder Societies with other National organizations of approximately similar standing and character might profitably set up some agency through which they might co-operate as occasion requires. Such an agency might well meet the following requirements: Its field of activity should be specific. It should have definite responsibility to its sponsors, yet with some freedom of initiation and action. It should be elastic enough to meet varying needs, to act promptly in emergencies, to enlist such aids as might be helpful. Its vision should be nation wide and its organization such that the influences initiating its action should be no less broad. Its chief function should be largely to bring about concerted action of various organizations as their mutuality of interest and influence might seem necessary, rather than to attempt to speak for them, and it should function so as to leave room for expression of minority opinion or qualified approval where unanimity could not be obtained except as a result of compromise. It should have no incentive to be active solely for the sake of activity. The dangers of a super-organization should be thoroughly guarded against by tying it closely to its sponsors, and it may well be that its organization could be composed of men serving automatically by virtue of their offices in the sponsoring societies. Such an agency, by whatever name it may be called, in addition to being useful in co-operative efforts, would doubtless in large measure promote the welfare of each of the sponsoring societies. Surely, of all men, engineers should be able to construct an efficient, adaptable, and enduring organization which would foster the spirit of co-operative effort, making it an agency through which the Engineering Profession, in those larger matters in which it is and should be concerned, may be of greater force and effectiveness.

The profession of engineering has certain obligations and responsibilities which its fellow members owe to each other and to the community and the State; these, in most cases, can be best discharged by the individuals through a professional society such as ours. The profession of Civil Engineering has certain rights and privileges which it may properly claim from its fellow members and from the community and the State; these too, in most cases, can be obtained most effectively by the individuals through a society such as this. This Society in the past has been interested in those things which have influenced the standing, the proficiency, and the welfare of the engineer. If its growth and prosperity are to continue, and in even greater degree, then there must follow, especially in these days of society competition, a still greater interest and activity by the Society in all matters that pertain to and concern the welfare of the professional engineer and those matters of public concern in which the engineer can aid in securing a wise solution. The Society can only accomplish its aims and purposes through the efforts of its membership. The need, therefore, is for a continued and increased interest and loyalty on the part of all members.

MEMOIRS OF DECEASED MEMBERS

JOHN ANDERSON BENSEL, Past-President, Am. Soc. C. E.*

DIED JUNE 19, 1922.

John Anderson Bensel was born in New York City on August 16, 1863, He was the son of Brownlee Bensel, whose ancestors came from Bruges, Belgium. His mother was Mary Maclay Hogg, the daughter of John Anderson Hogg, a Scotchman and lawyer, who was associated with Charles O'Connor, a leading member of the New York Bar about fifty years ago. Mr. Hogg was considered by Mr. O'Connor to be among the most brilliant and promising of his young assistants. Mr. Hogg's wife was a Miss Maclay, the daughter of a prominent clergyman of Scotch ancestry. In this, Major Bensel took great pride, showing a love for all the old Scotch customs and holding the virtues and sturdy characteristics of his Scotch ancestors in high esteem. Thus, his membership in the St. Andrew's Society was always a source of pleasure, and, for many years, he took an active interest in all its functions.

Mr. Bensel was the eldest of eight children, and at the age of twelve had many responsibilities thrust upon him on account of the death of his father. A sketch of his early life is a reminder of the wonderful growth of New York City in the past half century. Born in 24th Street, his family moved, when he was six years old, to 155th Street in the vicinity of Audubon Park, which was, at that time, the country-side far beyond the city proper. From there, he attended school far down town, the Twelfth Street Public School, celebrated for having turned out many notable men; later, he attended the "Friends School" on East 13th Street. This connection with the Quakers conveyed some lasting impressions, and he always retained a great admiration for them, their standards and ideals, the simplicity of their dress, and the spirituality of their religious exercises. At about his sixteenth year, the family returned to the city proper and lived for many years at 64 East 79th Street. The gap caused by the loss of his father at an early age, was fortunately filled by a mother of unusual attainments. A strict disciplinarian, she brought her young family up in an atmosphere of the utmost simplicity. Possessed of a cultivated mind and high ideals, and of unusually intimate knowledge of history and literature, she was qualified to supply the lack in the ordinary public school curriculum extant at that time. Although this effort of his mother to transmit something of her knowledge of the classics did not outwardly bear direct results, the boy's mind running on purely practical lines, the impressions were lasting, and showed in his appreciation of everything beautiful. His artistic sense and his keen appreciation of beautiful things were not only evident in his house and its furnishings, but in every structure that he had a part in designing.

Mr. Bensel was graduated from Stevens Institute of Technology on June 12, 1884, with the degree of Mechanical Engineer. On June 21, 1921, his

* Memoir prepared by George H. Pegram, Past-President, Am. Soc. C. E., and John P. Hogan, Robert Ridgway, Charles W. Staniford, and William H. Yates, Members, Am. Soc. C. E.

Alma Mater conferred on him the honorary degree of Doctor of Engineering in recognition of his distinguished engineering service.

His first work after graduation was as Rodman in the Department of Public Works of New York City, where he was engaged on contour and property surveys for the proposed new Croton Dam.

On October 1, 1884, Mr. Bensel was appointed Rodman in the Maintenance of Way Department, United Railroads of New Jersey Division of the Pennsylvania Railroad, and, in 1886, he was promoted to be Assistant Engineer. During his connection with this Company, he made surveys of its holdings in New York City and Jersey City; superintended the repair of various piers and ferry racks; made plans and superintended the construction of a new pier and freight shed at 35th Street, North River; and of the following structures: The Williamsburg Terminal; the freight shed on Pier 3, Jersey City; car houses at Bayonne, N. J.; ferry structures at Bayonne, and at Port Richmond, Staten Island; and the steam-power, counterbalanced, float bridge at Harsimus Cove, Jersey City. He prepared a paper* for the Society in 1888 describing this float bridge. In September, 1888, he was promoted to be Assistant Supervisor, Division A, and given charge of the main-line yards and terminals between Jersey City and Newark, N. J.

Mr. Bensel resigned from the service of the Pennsylvania Railroad in 1889 to accept a position as Assistant Engineer in the Department of Docks, New York City. As Assistant Engineer from 1889 to 1896, he had charge of works of construction on the North River, including the Laight Street Section of bulkhead or river wall; Pier New 29; the sections of bulkhead walls at Franklin Street and West Washington Market; Pier New 23, and Piers New 14 and 15, for the American Line, which at the time were the most modern piers in the city, with an elaborate double-deck steel shed on Pier 14. The work at Pier 14 embraced a difficult piece of rock removal to a depth of 40 ft. below low water, which was successfully accomplished under his immediate supervision by a Department force. He described this work in a paper entitled "Removal of Rock 40 Ft. below Surface of Water, North River, N. Y.", which has been published by the Society.†

In 1896, he left the Department of Docks to become a member of the firm of Steers and Bensel and, for two years, conducted a large amount of dredging and pier construction in and about New York Harbor.

In 1898, Mr. Bensel was appointed Engineer-in-Chief of the Department of Docks, New York City, serving eight years until 1906, when he was made Commissioner of Docks by Mayor George B. McClellan, serving two years, until 1908, when he went to the Board of Water Supply as President.

Many notable works and improvements were initiated or carried out by Mr. Bensel during the time he served as Engineer-in-Chief and Commissioner of the Department of Docks, among which were the nine double-deck piers and sheds of the Transatlantic Steamship Terminal on the North River, known as the "Chelsea Section;" the extensive group of very long piers in South Brooklyn; the retaining wall enlargement of Riker's and South Brother

* *Transactions*, Am. Soc. C. E., Vol. XIX (1888), p. 309.
† *Transactions*, Am. Soc. C. E., Vol. XXXII (1894), p. 231.

Islands; extensive bulkhead wall and pier work on the North and East Rivers; and the construction of all the municipal ferries, consisting of four elaborate steel ferry-houses, eleven slips, and eight of the largest ferry-boats in New York Harbor.

During 1905 and 1906, while Engineer-in-Chief, he acted as Consulting Engineer on an extensive bulkhead wall project in Philadelphia, Pa., for the Girard Estate, for which his services were sought on account of his long experience with this difficult phase of construction in New York City.

Bulkhead wall construction, particularly around Manhattan, presenting many unusual problems in foundation work and involving, as it did, the expenditure of comparatively huge sums of money, had long been studied by boards of engineers and had constantly given concern to the engineers of the Department of Docks. Perhaps the study of this great problem, to which as Assistant Engineer, and, later, in control, Mr. Bensel gave the most lively interest, and which resulted in an adequate design at reduced cost, is a feature which will always link his name with the work of the New York Dock Department. Briefly, this 16-year period of his life was one continuous study to accomplish something worth while by leaving the beaten paths, and to do this, he in no way spared himself or his own comfort. It has been said that Mr. Bensel made more dock history for the epoch during which he was Engineer-in-Chief and Commissioner than any other individual for a similar period of time. He never believed in sacrificing money for a city improvement that was not in full keeping with the standard New York should have. He said, "no one appreciates a poor structure or utility."

Mr. Bensel was appointed a Commissioner of the Board of Water Supply by Mayor McClellan on January 31, 1908, and, on the same day, he was elected President of the Board by the other two Commissioners. He served in this capacity until his resignation on December 31, 1910, assuming on the following day the office of State Engineer and Surveyor of New York. His services were characterized by his tremendous personal energy, and his administrative ability effectively advanced the work of the Board.

The time during which Mr. Bensel was connected with the Board of Water Supply was a period of great activity, not only in the actual work accomplished, but, also, in the amount and number of contracts awarded. Forty-one contracts representing a construction cost of $54 000 000 were awarded during his term of office, and work to the value of more than $25 000 000 was done. As the contract cost of the first stage of the Catskill Water System for the delivery of at least 250 000 000 gal. per day from the Esopus water-shed was about $100 000 000, it is seen that during these three years contracts were awarded for more than half the work and one-quarter of it was actually done. Several important questions came up and were settled during this time, including the difficult problem of carrying the Catskill Aqueduct across the Hudson River at Storm King by means of the deep pressure rock tunnel, 1 100 ft. below the water surface. Another problem related to the type of structure for the aqueduct traversing the several Boroughs of New York City. Through Manhattan, The Bronx, and a part of Brooklyn, it was decided, on the recom-

mendation of the Chief Engineer, to use a deep, pressure, concrete-lined rock tunnel instead of the delivery pipes tentatively suggested in the original plan.

At the Democratic State Convention in the summer of 1910, Mr. Bensel was a prominent candidate for Governor of New York State. The Convention did not select him for this office, but nominated him as the candidate for State Engineer and Surveyor, to which post he was elected in the fall of 1910, taking office on January 1, 1911. He was re-elected in 1912, and thus served two terms of two years each. During his administration, work was actively advanced on the Barge Canal, the main line of which extends from the Hudson River to Lake Erie and was the most extensive public work ever undertaken by the State of New York. Although work on this improvement had been commenced under previous administrations, 60% of the total amount was performed during his terms of office.

One of Mr. Bensel's greatest achievements in connection with this project was the change in plan which combined Locks Nos. 2 and 3, of the Cayuga-Seneca Canal, at Seneca Falls, thereby increasing about 100% the horsepower available there for commercial purposes.

A Principal Assistant on his staff as State Engineer, writes:

"One of the things which impressed me greatly was the courageous way in which he handled his engineering appointments while State Engineer, which, like many other executive positions in the State service, are very largely of a political nature, and the pressure to appoint engineers to various positions because of their political affiliations is sometimes quite manifest. Mr. Bensel's attitude in handling the appointments was always very pleasing to the members of the profession. A man's standing as an engineer, his qualifications for the particular position under consideration or to be filled, his term of previous service and right to promotion always came first in Bensel's mind, and political consideration as secondary. This is best evidenced by his appointment of deputy and division engineers, and others, who were promoted to appointive positions carrying substantial salaries. In most instances, they were men of different political faith and were entirely unknown to Mr. Bensel previous to his assuming the duties of State Engineer and were appointed entirely on their past record and qualifications for the positions given them."

In 1915, Mr. Bensel opened an office in New York City and, thereafter, until his death, he was engaged as a consultant on many public undertakings, including the proposed hydro-electric municipal power-house at Oswego, N. Y.; the Great Western Gateway Highway Bridge over the Mohawk River, at Schenectady, N. Y.; and the proposed water-front improvements for Jersey City, N. J.

In 1919, he was appointed a member of the Board of Consulting Engineers for the Vehicular Tunnel under the Hudson River at Canal Street, New York City, connecting the States of New York and New Jersey, serving in such capacity until his death. The Chief Engineer has summed up Major Bensel's connection with this great work in the following manner:

"As a consultant, I found him always ready to give me the benefit of his years of experience in public work to such an extent that at times his interest might be said to have been almost paternal—an interest which I

found to be most enjoyable and helpful. His advice was particularly helpful when it came to broad matters of policy in regard to which he had had many years of experience in public life, and this experience was a reservoir of the greatest value. Major Bensel, as a consultant, had a perfect understanding of the relationship which he bore to me as Chief Engineer, which contributed in no small part to the pleasant relations which we enjoyed."

At the outbreak of the World War, Mr. Bensel immediately offered his services to the Government and, on May 16, 1917, he was commissioned a Major in the Engineer Officers Reserve Corps. He was assigned to duty at Norfolk, Va., where he remained until after the close of the war as Consulting Engineer on the port and harbor development at that Base. He was relieved from active duty on January 3, 1919, after nearly two years' active service, and remained in the Reserve Corps as Major of Engineers until his death. On October 30, 1919, he was assigned to the command of the 125th Battalion, Engineers, a Reserve unit.

Major Bensel died at his summer home, at Bernardsville, N. J., on June 19, 1922.

He was married on November 11, 1896, to Ella L. Day, the daughter of Henry Day and Adelaide Scofield Day. Mr. Day came from New England, while Mrs. Day's people were residents of New York City for four generations. Mrs. Bensel and four children, Louise Day, John A. Jr., Evelyn Adelaide, and Henry Day, survive him.

Few of Major Bensel's many friends really knew him, but those who enjoyed this great privilege will realize how difficult it is to portray his character clearly. He was known as an able administrator and a capable engineer, fearless and progressive. Much of his success came from his keen insight of human nature and few possessed his ability to pass quick judgment with accuracy. Impetuous, but always under control, he was firm in his adhesion to first judgments, but was remarkable in his willingness to listen to appeal and argument. Subordinates could always be certain that an opportunity would be granted them to defend their grievances before him, if they desired to do so. One of his characteristics was the extreme openness with which he dealt with superiors and subordinates. Having a high sense of honor and high ideals regarding professional etiquette and public service, he insisted that those who gave him a professional opinion should be qualified by education and experience to do so, and he expected from those associated with him the same loyal service to a cause that he gave himself. His subordinates were frequently surprised to learn of the intimate knowledge he had of the details of the particular project that might be under discussion, and he expected them to have at least equal knowledge of such details and to know the reason for taking each step.

An associate of many years' standing, writes of him:

"In all my experience, I have never known a man who controlled large forces to be held in such high esteem by the laboring men and mechanics under him. This love and respect did not always come from those who received help, but often after receiving discipline. It was his pleasure to use his power to help the unfortunate to help themselves."

Another friend states:

"An appeal to the heart of John Bensel always received a generous response; indeed too much of his time was spent helping others. No effort on his part was too great when a friend needed aid. This characteristic was made manifest continuously until his death. It mattered little what kind of assistance was needed, he never spared either his energy or his purse to assist a friend."

When informed of Major Bensel's death at its meeting of June 19, 1922, the Board of Direction of the Society passed the following preamble and resolution:

"The Board of Direction of the American Society of Civil Engineers records with deep regret the death of John Anderson Bensel, Past-President of the Society, who was a member for thirty-seven years, during eleven of which he served on this Board. His death severed strong ties of friendship and has left us with a deep and lasting sorrow.

"As member of this Board of Direction, he brought to its counsels qualities of leadership upon which we were accustomed to rely and which cannot be replaced—a life-long study of the problems of engineering, and an intuitive knowledge of men, a sound and matured judgment, founded upon natural wisdom and long experience. His mental grasp ignored all narrow considerations and reached out to the larger view. His generous nature rejected all selfishness in favor of the common good. His courage was of the quality that never faltered. But perhaps his most striking characteristic was that clear integrity which was not mere honesty, but an unerring instinct of fairness and of right.

"*Resolved:* That the Board of Direction of the American Society of Civil Engineers give expression of their heartfelt and enduring sorrow to Mrs. Bensel and to his children."

Major Bensel was a member of the American Society of Mechanical Engineers, the American Institute of Mining and Metallurgical Engineers, and the Institution of Civil Engineers of Great Britain. He belonged to the Engineers', Lawyers', Union and University Clubs, of New York City, the Fort Orange Club, of Albany, N. Y., the Nassau County (New York) Golf Club, and the Somerset Hills (New Jersey) Country Club. He was a member of All Souls Unitarian Church of New York City.

Major Bensel was elected a Junior of the American Society of Civil Engineers, on September 2, 1885, and a Member on March 4, 1891. He was a Director from 1899 to 1901, inclusive, a Vice-President in 1907 and 1908, and was President of the Society in 1910. As Director, Vice-President, President, and Past-President, he served on the Board of Direction for eleven years. In addition to the papers previously mentioned, he was the author of the following papers and reports which were published by the Society: "Observations on Dock Work in New York Harbor";* "Final Report of the Special Committee to Investigate the Conditions of Employment of, and Compensation of, Civil Engineers";† "Final Report of the Special Committee on Floods and Flood Prevention";‡ and "Address at the 42nd Annual Convention, Chicago, Illinois, June 21, 1910."§

* *Transactions*, Am. Soc. C. E., Vol. LIV, Part F (1905), p. 3.
† *Loc. cit.*, Vol. LXXXI (1917), p. 1207.
‡ *Loc. cit.*, Vol. LXXXI (1917), p. 1218.
§ *Loc. cit.*, Vol. LXX (1910), p. 464.

ROBERT MOORE, Past-President, Am. Soc. C. E.*

DIED JULY 24, 1922.

Robert Moore, the son of Henry Clay and Amelia N. Whippo Moore, was born at New Castle, Pa., on June 19, 1838. His great-grandfather, Dr. Henry Moore, came to the United States from Dublin, Ireland, in May, 1773, and settled in Washington, Pa. His maternal grandfather was Charles T. Whippo, an engineer employed on the construction of the Erie Canal, of Pennsylvania, who, later, became Chief Engineer of the Canal from Beaver to Erie, Pa.

Mr. Moore was graduated from Miami University, of Ohio, with the degree of Bachelor of Arts, on June 20, 1858, later, receiving the degree of Master of Arts. His early engineering training was obtained directly under his father, who was a prominent civil engineer, engaged largely in the pioneer railroad development in the Middle West.

Mr. Moore's early manhood covered the period of the Civil War. In 1863, he was engaged as a Civilian Engineer with the Engineer Corps, U. S. Army, on fortifications in Kentucky. From 1864 to 1877, he was engaged in the location and construction of various railways in Ohio, Indiana, Illinois, Missouri, and Kansas, lines which now form part of the trunk-line systems of the United States.

With the adoption of a new charter by the City of St. Louis, Mo., in 1876, the Board of Public Improvements, which has charge of the engineering work of the city, was established in 1877. Mr. Moore was a member of the first Board, and acted as Sewer Commissioner from 1877 to 1881. His confrères on this Board were all eminent men in the Engineering Profession, namely, the late Henry Flad, Past-President, Am. Soc. C. E., Thomas J. Whitman, M. Am. Soc. C. E., Gen. John W. Turner, and Mr. Charles Pfeiffer. The capable and efficient handling of public affairs by this Board gave the Profession a standing and prestige in St. Louis that has remained a factor in the government of the city to the present day.

Early in his professional career, Mr. Moore perceived the disadvantage under which a man labors, who becomes a one-company engineer, and, thenceforth, he devoted himself to consulting practice. The spirit of independence which this illustrates, was one of his most marked characteristics. Accordingly, from 1881 until his retirement from active practice in 1920, his engagements were very largely as a Consulting Engineer. During that time, he was connected with some of the most important construction projects and difficult problems of those years: He was in charge of the re-construction of the railway floor system of the Eads Bridge; Consulting Engineer of the St. Paul and Duluth Railway; Chief Engineer of the St. Louis Merchants Bridge Terminal Railway; and Consulting Engineer for the Re-organization Committees of the St. Louis Southwestern Railway, the Rio Grande Western Railway, and the Santa Fé System. He was Chief Engineer of the St. Louis, Peoria, and

* Memoir prepared by the following Committee appointed by the Board of Direction: Messrs. Baxter L. Brown, *Chairman*, F. G. Jonah, and W. E. Rolfe, Members, Am. Soc. C. E.

Northern Railway, a line extending from St. Louis to Peoria, Ill., from 1894 to 1898; and, during this time, he also represented the Los Angeles Terminal Railway in the San Pedro-Santa Monica Harbor controversy. He was also Chief Engineer of the Wabash, Chester, and Western Railway, and the Missouri Extension of the Missouri and Illinois Railway. Mr. Moore was a member of the Brazos River Board in 1897, and a member of the Southwest Pass Board in 1899 and 1900. He also acted as a Special Consulting Engineer on the Missouri and Mississippi River Bridges, for the Chicago, Burlington, and Quincy Railway, from 1900 to 1920.

In addition to his professional work during these years, Mr. Moore found time to take a most active part in the educational affairs of the City of St. Louis. He became a member of the Board of Education in 1897, at a time when the public had been aroused with respect to the evils existent in the public school system, and served as a member of what was then designated as the Reform Board, composed of twelve members, which took the place of the old system of ward representation. At that time, Mr. Moore had already reached an age when most men, although conscious of civic duty, feel entitled to be relieved from such demands on their time; however, he not only acted through his original term of election, but through successive terms, continuously, for sixteen years. He was President of the Board of Education in 1906 and, again, in 1910, and during practically the whole period of his service with the Board, when not holding the office of President, he was at the head of one of its important Standing Committees.

It was the time of the greatest improvement and development of the public school system in St. Louis, when it was brought from an exceedingly low ebb of inefficiency to a high tide of successful administration, taking its place easily as the best in the United States and the model for the work of other cities. It is not too much to say that this attainment was largely the result of Mr. Moore's quiet courage and intelligent labors.

Mr. Moore took a most active part in the ethical movement in St. Louis. He became Chairman of the Ethical Society in 1891, and, thereafter, until disabled by ill health, he remained as its active civic representative and its guiding spirit as Chairman Emeritus until his death. He worked for his creed with an unselfishness, a devotion, and a persistency worthy of the best traditions of his Puritan ancestors.

Mr. Moore's most marked characteristic was perfect mental integrity. He deceived no man, and, better still, he did not deceive himself. He cared nothing for praise or blame from the public, if he thought he was right. He had no ambition for himself. He was quiet, dignified, and reserved, a man of few words, with a marked sense of fairness and justice. This trait was so generally recognized that in all the work coming under his charge, covering a period of a great many years, he never had any disputes to litigate. His letters, papers, and addresses were models of clear thinking and concise diction.

During his long and active career, he was associated with, and had in his employ, many young men entering the Profession, whose success is due in a

large measure to the inspiration of his example, and whose association with him will always remain among their most cherished recollections.

Mr. Moore was married on October 3, 1878, to Alice Filley, of St. Louis, who, with a son, Charles Whippo Moore, survives him.

He was a Member of the Institution of Civil Engineers of Great Britain and one of the early members and a Past-President of the Engineers' Club of St. Louis.

Mr. Moore was elected a Member of the American Society of Civil Engineers on April 5, 1876. He served as a Director of the Society from 1892 to 1893, as Vice-President in 1888, and from 1889 to 1900, and as President in 1902.

"They cannot die whose lives are part
Of that great life which is to be,
Whose hearts beat with the world's great heart,
And throb with its high destiny.

"Then mourn not those who, dying, gave
A gift of greater life to man;
Death stands abashed before the brave;
They own a life he may not ban."

SIR DOUGLAS FOX, Hon. M. Am. Soc. C. E.*

DIED NOVEMBER 13, 1921.

Sir Douglas Fox was born at Smethwick, Birmingham, England, on May 14, 1840, and was the son of the late Sir Charles Fox and Lady Mary (Brookhouse) Fox. His father, Sir Charles Fox, was a pupil of and, subsequently, Assistant Engineer under, Robert Stephenson. He designed and erected the First International Exhibition of 1851 in Hyde Park, for which he was knighted, and, subsequently, for many years practiced as a Civil Engineer in Westminster, London.

Sir Douglas Fox was educated at Cholmondeley School, Highgate, King's College School, and King's College, London, of which he was a Fellow.

In 1860, he and his brother, Sir Francis Fox, joined their father in his business, the firm practicing as Sir Charles Fox and Sons. On the death of Sir Charles Fox, the name of the firm was changed to Sir Douglas Fox and Partners, and the firm, now located in London, has since continued to act professionally, both at home and abroad, as Engineers in Chief and Consulting Engineers to many important undertakings, among the earliest of which was the construction of the approaches of the London, Chatham, and Dover and the London, Brighton, and South Coast Railways to London.

In conjunction with the late Sir James Brunlees, the firm was associated with the constructional designs of the Mersey Tunnel between Liverpool and Birkenhead, for which work Sir Douglas Fox was knighted.

The firm was professionally connected with the constructional designs of the Swing Bridge, over the River Dee, on the Chester and Connah's Quay Railway at Hawarden, Cheshire; the Liverpool, St. Helens, and South Lancashire Railway; the Liverpool, Southport, and Preston Railway; the Liverpool Overhead Electric Railway which was the first electrified railway in England; the Snowdon Mountain Railway; the Cardiff Railway, for the Marquis of Bute; the Neath, Pontardawe, and Brynamman Railway, and the Southern Extension (Rugby to London) including the Marylebone Terminus, of the Great Central Railway.

With the late W. R. Galbraith, the firm acted as Engineers in the construction of the Charing Cross, Euston, and Hampstead Tube Electric Railway, which was opened in 1907. The firm also acted as the Engineers during the construction of the Great Northern and City Tube Electric Railway, from Finsbury Park to Moorgate Street, which was opened in 1904, and its members are Consulting Engineers to the Channel Tunnel Company, in connection with the proposal to construct a tunnel between England and France.

During the World War, the firm was associated with the design and construction of docks and slipways for Lord Furness, at Middlesbrough, the Explosives Factory for the Admiralty at Holton Heath, Dorset, and the Explosives Factory for Nobel's Explosives Company at Pembrey, South Wales.

* Memoir prepared by Sir Douglas Fox and Partners, London, England.

Sir Douglas Fox and Partners are engaged also as Engineers for important hydro-electric works in North Wales, for the Aluminum Corporation, Limited, and Joint Consulting Engineers, with Messrs. Livesey, Son and Henderson, to the Central Argentine Railway Company. With Sir Charles Metcalfe, the firm is acting as Joint Consulting Engineers to the Shiré Highlands (Nyasaland) Railway Company, the Rhodesia, Mashonaland, and Beira Railway Company (Cape to Cairo), the Benguella Railway Company, the Trans-Zambesia Railway Company, and to the British South Africa Chartered Company.

With the late Sir George Bruce and Mr. Robert White, the firm of Sir Douglas Fox and Partners was for many years Joint Consulting Engineers to the South Indian Railway Company, and is acting as Consulting Engineers to the Southern San Paulo Railway Company and the Dorada (United States of Colombia) Railway Company. The firm was responsible for the design of the steel bridge over the Zambesi River, at the Victoria Falls, in Rhodesia.

Sir Douglas Fox was constantly engaged as an expert engineering witness in the Parliamentary Committee Rooms at Westminster, and in connection with the promotion and opposition to bills for the construction of railways, harbors, etc., and the London tube railways.

He was interested in many companies. He served as Chairman of the British Griffin Chilled Iron and Steel Company, Limited, Barrow-in-Furness, the Improved Industrial Dwellings Company, Limited, Northfleet Coal and Ballast Company, Limited, Venezuela Telephone and Electrical Appliances Company, Limited, and The Thurrock Chalk and Whiting Company, Limited. He was a Director of the British South African Explosives Company, Limited, City of Las Palmas Water Company, Limited, Haematite Ore Concentrates, Limited, Pondicheri Railway Company, Limited, South Indian Railway Company, Limited, and United Electric Tramways Company of Caracas, Limited.

In 1921, he was presented with a testimonial by the Shareholders of the Northfleet Coal and Ballast Company, Limited, in recognition of his fifty years of service as Chairman, during which period he never missed an Annual General Meeting.

He was President of the Institution of Civil Engineers from 1899 to 1900 and, for many years, was an active member of its Engineering Standardization Committee. He was also a member of the Institution of Mining Engineers and the Institution of Electrical Engineers.

On May 26, 1863, he was married to Miss Mary Wright, daughter of the late Francis Beresford Wright, Esq., of Osmaston Manor, Derby. He is survived by five children, a son, Francis Douglas Fox, and four daughters. He was deeply interested in all Christian and philanthropic work.

Sir Douglas Fox was elected a Corresponding Member of the American Society of Civil Engineers on June 7, 1871, and an Honorary Member on March 5, 1901.

WILLIAM HASELL WILSON, Hon. M. Am. Soc. C. E.*

DIED AUGUST 17, 1902.

William Hasell Wilson, the son of John and Eliza (Gibbes) Wilson, was born at Charleston, S. C., on November 5, 1811. His grandfather was an Engineer and served as a British officer during the Revolutionary War. His father was also an Engineer and Surveyor, and served the United States as an Engineer Officer during the War of 1812. It is interesting to note that Mr. Wilson's grandfather planned the British attack on Charleston, S. C., and Savannah, Ga., during the Revolutionary War, and his father planned the American defense of those cities in the War of 1812.

Mr. Wilson received his education at Charleston, S. C., Morristown, N. J., and Philadelphia, Pa., where he entered the High School of the Franklin Institute. In June, 1827, he joined an Engineer Corps which his father had organized under the direction of the State of Pennsylvania, for canal and railroad surveys through Chester and Lancaster Counties. When this work was completed, he continued his studies, concentrating on drawing and mathematics.

In March, 1828, Mr. Wilson was employed as Rodman with a corps of engineers engaged in locating the Philadelphia and Columbia Railroad. In 1829, he became Assistant Engineer in charge of the construction of twenty miles of road in the Eastern Section, and held that position until April, 1831, when he was promoted to that of Principal Assistant Engineer in charge of construction of the entire Eastern Section, consisting of forty miles. The road was completed in October, 1834, and for a few years following, he was engaged in engineering work of a varied character.

In August, 1838, Mr. Wilson was appointed by the State of Pennsylvania as Chief Engineer of the Gettysburg Extension which was designed to connect the Philadelphia and Columbia Railroad with the Baltimore and Ohio Railroad. On the discontinuance of this work, in 1839, he returned to Philadelphia, and in February of that year he attended a meeting at Baltimore, Md., composed of engineers interested in organizing an engineering society. Plans were discussed, but it was finally decided that conditions were not then favorable for the success of the project, and the matter was dropped.

In 1839, Mr. Wilson was employed in making surveys for a proposed line from Downingtown to Reading, Pa., and also as Principal Assistant Engineer for the Philadelphia and Reading Railroad Company in the final location and construction of the Second Division of its road from near Pottstown to Bridgeport, Pa. The construction of the Black Rock Tunnel at Phœnixville, Pa., the entire length of which was excavated through solid rock, is a noteworthy part of this work. By the light of lanterns and candles, Mr. Wilson gave the instrumental work on this tunnel his personal attention. The excavation was made from both ends of the tunnel and through five shafts located at

* Memoir compiled from information furnished by Henry W. Wilson, M. Am. Soc. C. E., and on file at the Headquarters of the Society.

its edge, and when it was finished, the variation in alignment and grade did not exceed more than 0.01 ft. The fact that this work was undertaken sixty years ago, at a time when there was a dearth of geological data and construction tools, makes its successful completion very remarkable and worthy of note.

Owing to depressed business conditions, Mr. Wilson became engaged in farming and pursued this work with much zeal and interest from 1841 to 1852. Returning to the practice of his profession, he made surveys, during the summers of 1852 and 1853, for the Pennsylvania Railroad Company, and located a line from Philadelphia, *via* Phœnixville and the French Creek and Conestoga Valleys, to a point on the Harrisburg and Lancaster Railroad, about eight miles west of Lancaster, Pa.

In 1854 and 1855, Mr. Wilson served as Chief Engineer of the West Chester and Philadelphia Railroad Company. In January, 1856, he entered the service of the Pennsylvania Railroad Company, and made a survey for a railroad between the Delaware and Schuylkill Rivers through the southern part of Philadelphia. He also investigated railroad matters in Ohio, Indiana, and Illinois, in the interest of the Company.

In August, 1857, he was appointed as Resident Engineer of the Philadelphia and Columbia Railroad, which had been purchased by the Pennsylvania Railroad Company. He reconstructed a large part of this road, making many changes, among which was the relaying of 6 350 ft. of south track west of Dillerville, Pa., with entirely new rail, the last of the edge-rail track laid in 1834.

Mr. Wilson's jurisdiction as Resident Engineer was gradually extended, first to Mifflin, Pa., and by January, 1859, it included the entire line and branches between Philadelphia and Pittsburgh, Pa., with headquarters at Altoona, Pa. In 1862, his title was changed to that of Chief Engineer, and he was assisted by an Engineer of Bridges and Buildings and a Resident Engineer, on each of the Philadelphia, Middle, and Pittsburgh Divisions.

During the Civil War, Mr. Wilson was engaged in frequent consultation with the State and Federal civil and military authorities, to whom he gave able assistance on railroad matters of vital importance. The road being of strategic value, his was a position of great responsibility at this time.

In January, 1868, following a re-organization of his Department, which was necessitated by increased pressure of work, Mr. Wilson was made Chief Engineer of Construction, with headquarters at Philadelphia, where he was occupied in improving the various facilities of the Company, to provide for its growing business.

In November, 1873, he was elected President of the Philadelphia and Erie Railroad Company, at the same time continuing to act as Consulting Engineer of the Pennsylvania Railroad Company. In 1874, having been placed in charge of the Real Estate Department of the Pennsylvania Railroad Company, Mr. Wilson resigned his position as President of the Philadelphia and Erie Railroad Company. He was engaged in arranging and systematizing the work of that Department until his resignation in March, 1884, when he was again elected President of the Philadelphia and Erie, the

Belvidere-Delaware, the Philadelphia and Trenton, and other railroad companies controlled by the Pennsylvania Company.

In 1894, Mr. Wilson resigned all these positions except that of the Presidency of the Belvidere-Delaware Railroad and its subsidiary companies, which position he held until his death on August 17, 1902.

He was married on April 26, 1836, to Jane Miller, of Delaware County, Pennsylvania, who died on May 11, 1898. He is survived by five children.

It has been said of him that "always kind, charitable, and loving—a splendid type of the Christian man of science—his death, like his life, was beautiful, * * *. His work from the Delaware to the Ohio has left a lasting impression on the Commonwealth, and will remain as a monument to his abilities".

Mr. Wilson was elected an Honorary Member of the American Society of Civil Engineers on August 2, 1892. He was also an Honorary Member of the Engineers' Club of Philadelphia.

DECATUR AXTELL, M. Am. Soc. C. E.*

DIED NOVEMBER 27, 1922.

Decatur Axtell, the son of Almon and Sophronia (Boynton) Axtell, was born at Elyria, Ohio, on February 8, 1848. He was descended from a near relative of Daniel Axtell who was styled one of the twelve "Regicides", who condemned Charles I to death, ten of whom, including Daniel Axtell, were hung at Tyburn. Daniel Axtell was a Colonel in the Army of Oliver Cromwell and an intimate friend of Cromwell and his family. The Axtells, from whom Decatur Axtell was descended, were loyal to the Crown, and pronounced Daniel Axtell a fanatic; but the ignominy of Tyburn, and the disfavor of the name thereafter, drove one of the Royalist Axtells to join the "Pilgrim Fathers", which accounts for the Massachusetts ancestry of Decatur Axtell. His mother, through the Delanos, on her mother's side, traced her lineage to Philip de la Noix of France.

Mr. Axtell received his early education in Ohio, and during 1866 and 1867, was a student at Illinois College.

He began his railroad career on March 16, 1864, when he was engaged as Rodman on the Pacific Railroad of Missouri. In 1865, he was promoted to be Assistant Engineer of Construction on a division of the line south of Leavenworth, Kans.

After a year at Illinois College, Mr. Axtell became Engineer of Construction, Tunnel Division, of the St. Louis and Iron Mountain Railway, and held this position from October, 1867, to August, 1869. From the latter date until January, 1872, he served as Assistant Engineer of the same road and also of the Cairo and Fulton Railroad.

In January, 1872, he became Assistant Engineer of the St. Louis, Iron Mountain and Southern Railway and Chief Engineer of the Cairo, Arkansas and Texas Railroad, and held these positions until January, 1875. For a period of five years, until July, 1880, he served as Assistant Engineer and Division Superintendent of the St. Louis, Iron Mountain and Southern Railway.

In July, 1880, Mr. Axtell was appointed General Manager of the Richmond and Allegheny Railroad, which was then being constructed, and remained in that position until April, 1882, when he was appointed Vice-President and General Manager. He served in this capacity until June, 1883, when the Company was declared insolvent and he was appointed Receiver and Manager. At the end of his Receivership, in 1889, he was elected Second Vice-President of the Chesapeake and Ohio Railway, which Company had acquired the Richmond and Allegheny Railroad.

Mr. Axtell then began the task of rehabilitating the property of the Chesapeake and Ohio Railway Company. In February, 1900, he was appointed Vice-President, holding this position until February, 1918, when he retired. He was so closely connected with the work of the Chesapeake and Ohio Rail-

* Memoir prepared by James Poyntz Nelson, M. Am. Soc. C. E.

way Company that almost every part of the System bespeaks his prescience, ability, and professional masterliness.

He was also prominently connected with other railroads. From 1899 to 1903 he was President of the Toledo and Ohio Central Railway and Vice-President of the Kanawha and Michigan Railway. From 1903 to April, 1909, he was Chairman of the Board of Directors of the Toledo and Ohio Central Railway, and from 1903 to 1910 served in the same capacity with the Kanawha and Michigan Railway. In April, 1910, he became Vice-President of the Hocking Valley Railway, his services as such terminating when he retired as Vice-President of the Chesapeake and Ohio Railway.

Mr. Axtell died suddenly on November 27, 1922, at his home in Richmond, Va. He was married to Miss Ellen May Cantrell, a daughter of Dr. William A. Cantrell, of Little Rock, Ark., who survives him.

Mr. Axtell was known as a reticent man. He was called "the silent member of the Board of Directors." In business affairs, he moved with caution, realizing his responsibility, for on his judgment frequently rested the success of vast undertakings and the expenditure of large sums of money. Therefore, his advice had great weight with those who trusted him with the management of their affairs. He reasoned with meticulous exactness and was always ready to defend his conclusions, as well as to listen politely to the opinions of his opponents.

He was an engineer who thought in the finest categories of that profession and his methods bespoke his engineering training. He was also an able executive, with capacity for directing large enterprises; but, more than this, he was a gentleman who never failed in perfect courtesy, even under the most trying circumstances, a man who with utter directness ever gave the best that was in him and who, by a gentleness and courtesy all too rare, and with fine intelligence, made the obeyance of his orders a pleasure to those under him.

He was President of the Virginia Hot Springs Company from 1891 to 1911, and of the White Sulphur Springs Company from 1911 to 1918. He was also a member of the Virginia Historical Society, the South Carolina Historical Society, the Society of Mayflower Descendants, and the Society of Colonial Wars.

Mr. Axtell was elected a Member of the American Society of Civil Engineers on March 3, 1886.

ARCHIBALD STUART BALDWIN, M. Am. Soc. C. E.*

DIED JUNE 26, 1922.

Archibald Stuart Baldwin was born at Winchester, Va., on September 28, 1861, a son of Dr. Robert F. and Caroline (Barton) Baldwin. His family originated in Bucks County, England. The first Baldwin settlers in the United States came to New England in 1632 and were all kindred. His American progenitor was John Baldwin, who came to Milford, Conn., with the "New Haven Company". Among this family, Henry Baldwin was Judge of the Supreme Court of the United States, Mathias Baldwin established the famous locomotive works that bear his name, and others have been governors of States, members of Congress, generals of armies, divines, and authors.

Dr. Robert F. Baldwin, Mr. Baldwin's father, was a prominent practicing physician in Winchester until 1861, when he became Colonel of the 31st Virginia Regiment attached to "Stonewall" Jackson's Brigade. He was captured at the Battle of Rommey in 1862 and was confined as a prisoner of war in Fort Chase. After his exchange, he served as a surgeon in the Confederate Army until the close of the Civil War, when he was elected Superintendent of the Hospital for the Insane at Staunton, Va., occupying this position until his death in 1879. He was married to Caroline M. Barton, daughter of the Hon. Richard W. Barton, U. S. Congressman, and six children were born to them.

Archibald Stuart Baldwin was educated at the Shenandoah Valley Academy at Winchester and at the Staunton Military Academy. His desire was to enter the Medical Profession, in which his ancestors had been prominent for so many generations, but his father died when he was only seventeen years old, and he found himself in young manhood cut off from paternal advice and sympathies, facing the general prostration of after-war conditions, and dependent on his own efforts.

After teaching school for a year, he began his railway career as a Rodman on the Richmond and Allegheny Railroad (now a part of the Chesapeake and Ohio Railway); but in 1880 he entered the employ of the Iron and Steel Works Association of Virginia, with which Company he was engaged for three years on railroad and blast-furnace construction and the development of ore mines. The opinion has been expressed, by one who knew him well, that his varied experiences in connection with this Company contributed largely toward his subsequent success.

In 1883, Mr. Baldwin found employment on the extension of the Baltimore and Ohio Railroad into Philadelphia, Pa., as a Draftsman and Assistant Engineer, in which capacity he had charge of the construction of bulkheads and docks on the Schuylkill River. In 1885, he went West to accept a position with the Chicago, Milwaukee and St. Paul Railway Company as Principal Assistant Engineer on the construction of the Missouri River Bridge at Kansas City. The usual migrations of the younger members of the

* Memoir prepared by a Committee consisting of Charles F. Loweth, President, Am. Soc. C. E., D. J. Brumley, E. T. Howson, and John W. Alvord, Chairman, Members, Am. Soc. C. E.

Profession, however, found him after about a year a Resident Engineer on the construction of the St. Louis and Texas Railroad, now a part of the Louisville, Henderson and St. Louis Railroad, where he remained for a short time. On September 1, 1887, he was appointed an Assistant Engineer with the Louisville and Nashville Railroad and assigned to its development in the mineral district around Birmingham, Ala. Later, he was advanced to Principal Assistant Engineer in the office of the Chief Engineer at Louisville, Ky. He remained in this position until May 31, 1891, when, feeling that his health required outdoor work, he was transferred to that of Roadmaster on the main line, with headquarters at Elizabethtown, Ky., where he remained for nearly ten years, gradually building up his strength and maturing in experience and character.

On September 1, 1901, Mr. Baldwin left the service of the Louisville and Nashville Railroad Company to become Principal Assistant Engineer of the Illinois Central and the Yazoo and Mississippi Valley Railroads at Chicago, Ill. He was advanced to the position of Engineer of Construction on May 1, 1903, and promoted to be Chief Engineer on March 20, 1905. During his service as Chief Engineer of these railroads, the properties went through a period of great development in both construction and maintenance work; numerous branches and auxiliary lines were built, and, on many parts of the line, grades were reduced, alignment was improved, and second track was built, a number of large yards were constructed, terminal facilities at New Orleans, La., St. Louis, Mo., and other places, were enlarged, and the general condition of the roads was greatly improved.

Mr. Baldwin held the office of Chief Engineer of the Illinois Central Railroad until August 1, 1918, when he was elected Vice-President of the Corporation, which position he held while the railroad was under Federal Control. At the conclusion of Federal Control, on March 1, 1920, he was re-elected Vice-President and was placed in charge of the Chicago Terminal Improvement of the Company.

This improvement was involved with the Chicago Plan, calling for a lake front park skirting the South Side, with viaducts above and subways below the railroad tracks, giving access to it, and for the creation of a harbor district and dock facilities from 16th to 31st Streets, with railroad connections thereto. The Railroad Company's projected improvements include the reconstruction of its terminal north of 51st Street; the building of a passenger station fronting on Roosevelt Road, adjacent to, and in architectural harmony with, the Field Museum of Natural History; the building of a new suburban terminal at Randolph Street; the rebuilding of the South Water Street Freight Terminal; and the electrification of the entire terminal to Matteson, 28 miles south of the Chicago River; and the South Chicago and Blue Island Branches.

These vast projects, requiring years to build, had to be thought out and carefully negotiated among many conflicting interests. Here was a public service of the first order, although apparently done in the line of duty. It took more breadth of view and regard to the public welfare than any narrow conception of corporate interest to conclude this great public benefit to the satisfaction of every one. The negotiations were begun in 1912 between the City of Chicago, the South Park Commissioners, and the Illinois Central

Railroad, and, ably supported by President Markham, Mr. Baldwin handled the entire matter for the Illinois Central to its successful conclusion in the Contract Ordinance of July 20, 1919.

Before arriving at a decision on many important questions arising in connection with the improvement problem, investigation of European electrifications and passenger terminals was deemed to be desirable. Opportunity for such an inspection trip was afforded at the conclusion of the sessions of the Ninth Congress of the International Railway Association, held in Rome, Italy, in April, 1922, which Mr. Baldwin attended as Official Reporter on the subject "Terminal Stations for Passengers". It was while returning from this trip that he died suddenly of heart trouble on a train near Detroit, Mich. The funeral was held at the Hyde Park Presbyterian Church, Chicago, on June 29, 1922, and interment was at Staunton, Va., on June 30, 1922.

Mr. Baldwin was a staunch supporter of technical societies, and an active worker in them. Immediately on coming to Chicago in 1901, he became a member of the then newly formed American Railway Engineering and Maintenance of Way Association (now the American Railway Engineering Association), and the heartiness with which he threw himself into the activities of this Association is shown by his prominence on its important committees, such as the Committee on Signs, Fences, and Crossings; the Committee on Rail; and the Committee on Stresses in Track. He was also a Director of the Association in 1911, Vice-President in 1914 and 1915, and was honored with the Presidency in 1916, subsequently serving the usual five years as Past-President on the Board of Direction.

Mr. Baldwin became a member of the Western Society of Engineers in 1902. In 1918, he was elected Third-Vice-President, and made Chairman of the Finance Committee and a Member of the Amendments Committee. He was elected President of the Society in 1919, and on assuming this office brought a broad program of increased activities to the attention of the Board and the Society. In this regard, Edgar S. Nethercut, M. Am. Soc. C. E., Secretary of the Society, can best be quoted:

"A development committee was appointed by Mr. Baldwin to consider the problem. On this committee were men experienced in society work and on whose judgment the Board could rely. The recommendation that the society undertake a membership drive was considered at a special meeting of the members of the board, chairmen of committees and active workers. As chairman of this meeting Mr. Baldwin encouraged the widest discussion, after which he summed up the various arguments and proposals which had been offered, concluding with the recommendation that we undertake such a drive. Through his influence the different points of view were united and the vote finally taken was unanimous. The subsequent enthusiastic support of the campaign by the members was largely due to his leadership and to his marked ability and uniform courtesy. At the conclusion of the drive the Society was confronted with the problem of assimilating a 200 per cent. increase in membership and to this work, as president and continuing as a member of the Board of Direction and past-president, his thought was most constructive."

Mr. Baldwin was also active in important committee work of the American Railway Association, serving on the Committee on Direction, the General Committee, and the Committee on Third Rail and Overhead Working Conductors of its Engineering Division.

During the World War, he was necessarily tied more closely than ever to the work of his Railway Company. Nevertheless, he found time to serve as a member of the War Board of the Engineering Societies of Chicago, giving freely of his time to that important work. At the suggestion of Engineering Council, he was appointed a member of the Advisory Committee of the Federal Board of Surveys and Maps in 1920.

In addition to the societies mentioned previously, Mr. Baldwin was affiliated with a number of other societies and clubs, including the following: Society of Cincinnati of the State of Virginia, American Railway Guild, Franklin Institute, Chicago Engineers' Club and the Flossmoor Country Club. He was also an Honorary Member of Tau Beta Pi.

Mr. Baldwin was raised in the Protestant Episcopal Church, but, later, became a member of the Presbyterian Church, of which his wife was a member, and of which he was a faithful and devoted attendant and adherent. At the time of his death, he was a member and Elder (since April, 1908) of the Hyde Park Presbyterian Church of Chicago, materially aiding the church by his wise judgment and constructive ability. He was also for many years a member of the Board organized to promote and build the Presbyterian Home for the Aged and Helpless. He entered on his duties as Chairman of the Building and Grounds Committee at a time when the plans of the main building in Evanston were being drawn.

Mr. Baldwin's professional labors, spent entirely in building the nation's transportation system, cover a period of more than forty years. It was a life devoted to the common good, with little regard for personal reward. He was intensely interested in upholding the high ideals of the profession with which he had allied himself, as was illustrated in a manner so characteristic of him in an address, "The Engineers' Trust", which he delivered at the Annual Dinner of the Western Society of Engineers* on the occasion of his retirement as President of that Society. His sympathy was particularly with the younger man. This is revealed in a striking manner in an address which he delivered on "The Young Engineer", before the students of the University of Illinois in April, 1918. With his own early experience undoubtedly in mind, he spoke feelingly of the problem of selecting a vocation when age and experience seem inadequate for so important a decision. This address is summed up in the following quotation:

"What is judgment? It is that faculty of mind formed by a combination of education, experience, and character, that guides into understanding of principles involved and leads to correct decision. Education is the cultivation of inherent powers by discipline and instruction. Experience is another form of education which comes from actually doing and seeing, and is often so persistently repeated as to cause automatic action of mind without process of thought, and is valuable because of its sureness, but when repeated beyond a certain point has no further educational advantages.

"Character is the resultant of the forces of the mind, the spirit, and the body. The spirit is the God-like portion of a man, and the condition of the body is of no small importance, for weakness or ill health, in addition to incapacitating, may often times cause a lack of boldness in seizing a psychological moment.

* *Journal*, Western Soc. of Engrs., March 5, 1920.

"Thus judgment becomes the most valuable possession of the individual, and the net result of attainment will be largely governed by the measure of its growth."

Of all Mr. Baldwin's personal qualities perhaps the one which was most outstanding was his modesty. One is often compelled to admire ability in men who fully appreciate it themselves. Often, again, modesty is worn as if it was an ill-fitting garment, obligatory through custom, but compelling unnecessary obscurity. Mr. Baldwin's modesty was innate and unconscious. It expressed itself in the atmosphere he created without his knowing anything about it. It came out of the heart from the culture of generations. There is nothing about a man so lovable to men as this unconscious modesty.

Among his papers found after his death was a pocket notebook in which, among engineering data, he had copied the following quotations:

"To set an example of abstinence from petty personal controversies, and of toleration for everything except lying."

* * * * * * * *

"To be indifferent as to whether the work is recognized as mine or not as long as it is done."

* * * * * * * *

"Never let a man imagine that he can pursue a good end by evil means without aiming against his own soul; any other issue is doubtful; the evil effect on himself is certain."—SOUTHEY.

* * * * * * * *

"If thou art worn and hard beset
With sorrows that thou wouldst forget;
If thou wouldst read a lesson that will keep
Thy heart from fainting and thy soul from sleep,
Go to the woods and hills: No tears
Dim the sweet look that Nature wears."

* * * * * * * *

"E'en as he trod that day to God, so
Walked he from his birth,
In simpleness and gentleness and
Honour and clean mirth."—KIPLING.

On December 19, 1883, Mr. Baldwin was married to Miss Martha Frazier, of Staunton, Va., who, with one daughter, Katherine M., and three sons, Robert F., Howard F., and W. Frazier, survives him. He is also survived by two sisters, Mrs. Hugh C. Preston and Mrs. Barton Myers, and three brothers, Robert Frederick, William Barton, and John M., all of Norfolk, Va.

Mr. Baldwin was elected a Member of the American Society of Civil Engineers on December 6, 1905. He served on the Committee which worked out the new Constitution that was adopted by the Society in 1921, on the Nominating Committee, on the Washington Award Commission, and as a representative of the four Founder Societies at the Educational Conference at Montreal, Que., Canada. As a member of the Special Committee of the Society, to Report on Stresses in Railroad Track, he had much to do with the unusually original and important work of that Committee on this (to the railroads) vital subject. At the time of his death, he was a candidate without opposition for Director from District No. 8.

CHANNING MOORE BOLTON, M. Am. Soc. C. E.*

DIED DECEMBER 11, 1922.

Channing Moore Bolton, the fourth son of Dr. James and Anna Maria (Harrison) Bolton, was born at Richmond, Va., on January 24, 1843. His father, Dr. Bolton, was born in Savannah, Ga., in 1812, and was a graduate of Columbia College and the College of Physicians and Surgeons of New York City. He afterward studied theology and was ordained as a clergyman of the Protestant Episcopal Church, but returned to the practice of medicine in Richmond after a brief period spent in the ministry. The Bolton family is of English origin. Its pedigree has been traced to a period following the Norman Conquest when the family possessed large estates in Yorkshire and Lancashire.

Channing Moore Bolton received his early education at several private primary schools and at the preparatory school conducted by Mr. William D. Stuart in Richmond. In 1860, he entered the University of Virginia where he studied Latin, French, and mathematics. In 1861, he joined one of two student companies from the University that entered the Confederate Army, and was engaged in constructing fortifications around Richmond and in charge of building three forts near Brooke Turnpike.

In 1862, he was employed on the construction of a railroad to fill a gap between Danville, Va., and Greensboro, N. C., and continued in the railway engineering service holding the positions of Rodman, Transitman, and Resident Engineer in the Confederate Army. In the spring of 1863, he was commissioned a Lieutenant of Engineers in the First Regiment of Engineer Troops and ordered to report to Major-General Pender, of A. P. Hill's Army Corps, to act as Engineer Officer on his Staff. He joined General Pender at Winchester, Va., and accompanied him on the campaign into Pennsylvania where he took part in the Battle of Gettysburg.

Lieutenant Bolton assisted in the construction of the pontoon bridge across the Potomac River over which the Confederate Army crossed on its return to Virginia. He was in charge of the bridge when the crossing took place and also caused it to be destroyed when the Confederate troops were safely over, just before the arrival of the Federal forces. He took part in most of the battles fought by the Army of Northern Virginia.

Mr. Bolton continued in the Army until the end of the war in 1865, and at this period suffered many hardships and privations. After a few months spent in Southwestern Virginia, he returned to Richmond. In 1866, he surveyed, located, and constructed the Clover Hill Railroad in Virginia. In the latter part of 1866, and in 1867, he was engaged in constructing the tunnel at Richmond for the Richmond, Fredericksburg, and Potomac Railroad, and from 1867 to 1869 he was Resident Engineer of the Louisville, Cincinnati, and Lexington Railroad.

From 1869 to 1874, he was Engineer of the Chesapeake and Ohio Railway in charge of construction of a part of the road between Covington, Va., and

* Memoir compiled from information on file at the Headquarters of the Society.

White Sulphur Springs, W. Va. This was one of the heaviest pieces of railroad work constructed to that time. Mr. Bolton then organized and was in charge of a party of engineers, which located the road down New River in West Virginia, a very difficult enterprise. After locating the line from Richmond to Newport News, Va., he located and constructed a double-track tunnel, ¾ mile long, under Church Hill, Richmond, which arduous undertaking was successfully accomplished.

From 1876 to 1879, Mr. Bolton was United States Assistant Engineer in charge of the canal around the Cascades of the Columbia River in Oregon, and designed all the plans for this work. In 1879 and 1880, he was Division Engineer of the Richmond and Allegheny Railroad and he also had charge of the location and construction of that road from Richmond to Lynchburg, Va., which involved the changing of the old James River and Kanawha Canal into a railroad.

From 1880 to 1881, he served as Engineer and Superintendent of the Greenville (Miss.), Columbus and Birmingham Railroad, and from 1882 to 1895, he was Chief Engineer of the Richmond and Danville Railroad and of the Southern Railway.

During 1879 and 1880, Mr. Bolton was President and Manager of the Richmond City Street Railway which included all the street railways in Richmond at that time and of which he finally became sole owner. He operated these lines with great efficiency and eventually sold them at five times the original cost.

Mr. Bolton resigned as Chief Engineer of the Southern Railway in 1895, and moved from Washington, D. C., to his farm in Albermarle County, Virginia, engaging in private practice as a Consulting Engineer. In May, 1907, under contract, he built two tunnels near Garrison, Mont., one a double-track tunnel for the Northern Pacific Railroad Company, and the other for the Chicago, Milwaukee, and St. Paul Railroad Company.

Among his numerous activities may be mentioned that of President of the Charlottesville Street Railway Company and Charter Member and Director of the Charlottesville Ice Company. He was also President of the Meadow Creek Country Club, Trustee of the Miller School Board, member of the Miller Board of the University of Virginia and of the Executive Committee of the University of Virginia Alumni Association, and a Trustee of St. Paul's Chapel, University of Virginia. He drew the plans and supervised the construction of the beautiful Church of Our Saviour at Rio, Va., of which he was also a Trustee.

In 1911, Mr. Bolton became Director of the People's Bank of Charlottesville, Va., and in 1913, he was elected President of the Miller Board of the University of Virginia. He was also President of the Board of Trustees of St. Anne's School at Charlottesville and Chairman of the Highways Committee of the Charlottesville Chamber of Commerce. During the World War, he served as Chairman of the Local Board. In every capacity, he worked for the good of this locality.

Mr. Bolton contributed papers and reports to the Reports of the Chief of Engineers of the United States Army for the years 1877, 1878, and 1879.

He was married on February 17, 1874, to Miss Lizzie Calhoun Campbell, of Richmond, who died on October 6, 1889. His second wife, whom he married on June 6, 1894, was Miss Alma Ann Baldwin, of Montgomery, Ala. He is survived by his widow, his daughters, Mrs. J. Thompson Brown, Jr., of Rock Hill, S. C., Mrs. W. Allan Perkins, of University, Va., and Miss Cecile Baldwin Bolton, his son, Channing Moore Bolton, and his brother, Dr. Meade Bolton.

His constructive ideals and deep interest in the welfare of the community in which he lived will stand as a memorial of his worth, and the accomplishments of his vast energy are an index of his desire to serve his fellow men. Nothing deterred him from unswerving fidelity and faithful service to the cause of the Protestant Episcopal Church, and devotion to his family and loyalty to his friends were characteristics of his true personality.

Mr. Bolton was elected a Member of the American Society of Civil Engineers on January 4, 1888.

HENRY PERCY BORDEN, M. Am. Soc. C. E.*

DIED OCTOBER 19, 1922.

Henry Percy Borden was born at Port La Tour, Nova Scotia, on December 8, 1872. He attended Mount Allison University, at Sackville, New Brunswick, and, later, McGill University, at Montreal, Que., Canada, having been graduated in Civil Engineering by the Faculty of Applied Science of that institution in 1902.

Following his graduation, Mr. Borden entered the Engineering Department of the Canadian Pacific Railway, where he was engaged on the design of steel bridges and masonry and the inspection of the fabrication and erection of steel bridges, until 1904. He then accepted a position with the Montreal Locomotive and Machine Company as Assistant Engineer, in the Structural Department, on estimating and design, and, later, as Assistant to the Chief Engineer, in charge of the Estimating and Sales Department.

In 1906, Mr. Borden returned to the Canadian Pacific Railway as Architectural Engineer, in which capacity he had charge of the steel and reinforced concrete design for the Company's buildings, under the Chief Architect.

In 1908, he was appointed Secretary and Assistant Engineer to the Board of Engineers of the Quebec Bridge, which position he held until 1915, when he was made Assistant to the Chief Engineer of the same Board, and had charge of the designing office, mill, shop, and field inspection, under the Chief Engineer. After the death, in 1916, of Charles C. Schneider, Past-President, Am. Soc. C. E., one of the members of the Board, Mr. Borden was appointed to fill the vacancy thus created, which position he retained until the successful completion of the bridge in 1918.

Following the completion of the Quebec Bridge, Mr. Borden moved to Ottawa, Ont., Canada, entering the office of the Consulting Engineer to the Dominion Government, as Bridge Engineer. He retained this position until 1921, when he opened an office in Ottawa and began private practice as Consulting Engineer, specializing in bridges and structures of steel and reinforced concrete.

Mr. Borden was married in 1906 to Edith Eva Hall, of L'Orignal, Ont., Canada, who, with two daughters, survives him.

He was a member of the Engineering Institute of Canada, the Country Club of Ottawa, the Royal Ottawa Golf Club, and the Beaconsfield Golf Club, of Montreal.

Mr. Borden was elected a Member of the American Society of Civil Engineers on May 13, 1918.

* Memoir prepared by C. N. Monsarrat, M. Am. Soc. C. E.

BYRON HARKNESS BRYANT, M. Am. Soc. C. E.*

DIED JULY 8, 1922.

Byron Harkness Bryant, the son of Emory D. and Symanthy P. (Balou) Bryant, was born at Woonsocket Falls, R. I., on July 25, 1847.

He received his education at Kalamazoo, Mich., and began his engineering work as Rodman with a locating party on a survey for a railroad from Litchfield, Mich., to Fort Wayne, Ind., during May and June, 1867.

During July and August and until September 9 of that year, he served as Head Chainman on the location of the Peninsula Railway from Battle Creek, Mich., to Valparaiso, Ind. From September 9, 1867, to April, 1871, he filled the positions of Rodman, Assistant Division Engineer, and Division Engineer on the construction of the Grand Rapids and Indiana Railway.

From April, 1871, to January, 1872, Mr. Bryant was engaged as Chief Engineer of the Grand Rapids and Holland Railroad and, during 1872, he was Chief Engineer of the Middle Division of the Detroit and Bay City Railroad. From March to August, 1873, he was employed as Locating Engineer on the Atlantic and Great Western Railway, and from August, 1873, to October, 1879, he served as Chief Engineer and Superintendent of the Chicago, Saginaw and Canada Railroad.

Mr. Bryant held the position of Locating and Resident Engineer on the Denver and Rio Grande Railroad from October, 1879, to March, 1884. During this time, he located long sections of the railroad, made numerous explorations, including one through the Grand Canyon of the Gunnison River, out of which the famous irrigation tunnel has been constructed, and also had charge of the construction of 400 miles of road.

From March, 1884, until December, 1885, he was employed as Construction Engineer by the Canadian Pacific Railway Company, in charge of the construction of the line on the Lower Kicking Horse River, down the Columbia River, a section several miles in length over the summit of the Selkirks, and another section through the Gold Range Mountains. From March, 1886, until the end of that year, he held the position of Locating Engineer on the Montana Central Railway.

Mr. Bryant was Chief Engineer, Superintendent, and General Superintendent of the Colorado Midland Railroad, and Chief Engineer of the Busk Tunnel Railway from January 1, 1887, to December, 1904. While in this position, he built the Busk Tunnel, 9 400 ft. long, then the second longest tunnel in the world. From December, 1904, to July, 1905, he was Chief Engineer of the Colorado and Eastern Railroad.

His work was not confined to the United States. From July, 1905, to March, 1909, he was Chief Engineer of the São Paulo and Rio Grande Railway, with headquarters at Parana, Brazil. While in this position, he located more than 1 000 miles of line. From Brazil, Mr. Bryant went to Mexico, where he was appointed Chief Engineer of the Mexico-North Western Railway, at Chihuahua, and located and explored several hundreds of miles of line.

* Memoir prepared by Charles Hansel, M. Am. Soc. C. E.

Mr. Bryant left Mexico for Central America where, as Chief Location Engineer for the International Railways of Central America, he located projected lines in Guatemala, Salvador, and Honduras.

His great ability as a railroad location engineer was known and appreciated. He was a man of splendid character and ability, tireless in energy, thoughtless of self, always a gentleman, having kept a holy faith in God and his fellow man. His professional achievements were varied and important. Work was his happiness, achievement his compensation; he loved his profession and he was loved by all who knew him.

Mr. Bryant was elected a Member of the American Society of Civil Engineers on September 6, 1910. He was also a member of the Western Society of Engineers.

GEORGE BARKER BURBANK, M. Am. Soc. C. E.*

DIED FEBRUARY 29, 1920.

George Barker Burbank was born in Taylorsville, Ky., on March 16, 1844. His parents were of New England stock; his father, Moses Burbank was a native of Salem, Mass., and a graduate of Waterville College (now Colby College), Maine, and his mother, Nancy Baker, came from Lowell, Mass.

At the outbreak of the Civil War, George Barker Burbank, then a lad of sixteen, was living in Ludlow, Vt. Misrepresenting his age, he enlisted in Company I, Second Vermont Infantry. He was in all the battles of the Army of the Potomac from the first battle of Bull Run to that of the Wilderness. For exemplary conduct during the Battle of Fredericksburg, he was mentioned in orders and granted a furlough home. He was seriously wounded while serving as a Sergeant, at the Battle of the Wilderness, and his discharge followed his recovery. Mr. Burbank retained his intense patriotism throughout his life, and it was a keen satisfaction to him that his son, Clinton Montrose Burbank, Lieut., 2d Cavalry, U. S. A., served, with honorable mention and wounds, during the World War, half a century after his own military service was past.

From 1866 to 1884, Mr. Burbank practised his profession, chiefly railroad building, in the South and West. In 1885, he accepted an appointment as Assistant Engineer for the New York Aqueduct Commission, and, thereafter, with a few brief interludes, he resided in or near New York City, making his home for many years in White Plains, N. Y.

The following is a brief list of the more important engineering works with which Mr. Burbank was actively identified:

From 1867 to 1869, he was with the Louisville, Cincinnati and Lexington Railroad in Kentucky, filling all grades from Rodman to Engineer of Repairs, and from 1869 to 1870, he served as Division Engineer and Assistant to the Chief Engineer of the Cincinnati Southern Railway in Kentucky.

From 1870 to 1873, he was engaged as Division Engineer of the Wisconsin Central Railway, and from 1873 to 1874, as Resident Engineer of the Chesapeake and Ohio Railway.

From 1874 to 1875, he served as Division Engineer of the Cincinnati Southern Railway in Tennessee.

From 1876 to 1880, he was employed as Engineer, with the Eouhardt and Aurora Mining Company, Limited, in Nevada, and from 1880 to 1884, as Division Engineer and Deputy Chief Engineer, with the Rio Grande Western Construction Company.

Mr. Burbank was connected with the New York Aqueduct Commission, from 1885 to 1890, as Assistant Engineer and Engineer of Dam Construction, and from 1890 to 1894, he was employed by the Cataract Construction Company, of Niagara Falls, N. Y., as Resident Consulting and Chief Engineer. He was also engaged by the National Contracting Company, of New York City, from 1900 to 1903, as Consulting Engineer, and served the York Haven Power and Paper Company in a like capacity from 1903 to 1906. From 1911

* Memoir prepared by William Mayo Venable, M. Am. Soc. C. E.

to 1913, he was connected with the United Comstock Pumping Association and Siskiyou Mines Company, in Nevada and California, as General Manager.

In his general practice as Consulting Engineer, Mr. Burbank also investigated and reported confidentially on many other projects, with the execution of which he was not identified. This work took him to Porto Rico and Alaska, as well as to various parts of the United States.

Mr. Burbank continued in the active practice of his profession almost until the day of his death. The spirit of adventure which influenced the boy of sixteen to enlist, and a certain natural hardiness which made a pleasure of roughing it, characterized him throughout his active career. In the winter of 1906-07, at the age of sixty-two, he made a trip of 800 miles through the wilds of Alaska by dog team.

He is survived by his wife, a son, Clinton Montrose Burbank, and a daughter, Marjorie Church Burbank.

He was a Mason, and, at one time, Commander of Ludlow (Vt.) Post, G. A. R.

Mr. Burbank was elected a Member of the American Society of Civil Engineers on July 4, 1888.

SHIRLEY CARTER, M. Am. Soc. C. E.*

DIED JANUARY 5, 1923.

Shirley Carter was born in Richmond, Va., on January 4, 1869. He was a lineal descendant of the original Carters, who settled in Virginia early in the Seventeenth Century.

He first entered the Government service on August 16, 1888, serving in various grades in the Baltimore, Md., District of the Engineer Department, United States Army, until October 19, 1892. From October 25, 1894, to November 30, 1895, he was employed in various capacities in the Wilmington, Del., District. From September 23 to December 7, 1896, he appears on the rolls of the Montgomery, Ala., office. He then worked in a private capacity until October, 1902, on which date he re-entered the Federal service in Norfolk, Va., and served continuously in this District up to the time of his death. On January 1, 1907, he was promoted to the final and highest grade in the service, that of Assistant Engineer.

At the time of his death, Mr. Carter had immediate charge of all the river and harbor improvements in Norfolk Harbor and tributaries, the Inland Waterway from Norfolk to Beaufort, N. C., the improvement of Thimble Shoals Channel, and of the Nansemond and Pagan Rivers. These projects involved the expenditure of many millions of dollars, and their success has been due in no small measure to Mr. Carter's energy, foresight, good judgment, integrity, and unswerving loyalty to duty. He also supervised the operation and repairs of all floating plant in the Norfolk District. In this work he was especially capable and well qualified, on account of his long experience in the operation and repair of floating plant of all kind.

To quote from one who knew him well, as a friend and as a co-worker:

"Mr. Carter was considered by all the employees who had worked with and under him as a capable, hard working, conscientious engineer; was a man who attended strictly to his own business, and was always ready and willing to lend a helping hand to his fellow man."

His early death removes from the public service a loyal, rugged, and true-hearted employee, and leaves with those who were privileged to enjoy his friendship a keen sense of the loss of a friend who never failed in time of need.

Mr. Carter was elected a Junior of the American Society of Civil Engineers on May 31, 1892; an Associate Member on October 5, 1898; and a Member on December 4, 1906.

* Memoir prepared by J. P. Jervey, City Mgr., Portsmouth, Va.

WALTER LINSLEY COWLES, M. Am. Soc. C. E.*

DIED DECEMBER 4, 1922.

Walter Linsley Cowles, one of three sons of Robert Henry Cowles and Harriet M. (Beadle) Cowles, was born in Wallingford, Conn., on January 7, 1859. His mother was also a native of Wallingford. His father was born in Farmington, Conn., but spent most of his life in Wallingford, where he was engaged in the silverware business. He retired from active work as Treasurer of the Dime Savings Bank of that place. Robert Henry Cowles was seventh in direct line from an ancestor, John Cowles, who came to this country in 1635 and settled in Massachusetts, afterward removing to Hartford, Conn., and then to Farmington, Conn., in 1640.

Walter Linsley Cowles was graduated in 1878 from the Civil Engineering Course at Sheffield Scientific School, Yale University, for entrance to which he had prepared at the Wallingford High School. After his graduation, he became Designing Engineer with the John Clarke Company, Bridge Contractors, at Baltimore, Md. From 1881 to 1883, he was a member of the firm of the Clarke Bridge Company, also at Baltimore, and in 1883 and 1884, he was engaged in the contracting business with A. J. Twiggs and Company, Augusta, Ga. In 1884 and 1885, he served as Bridge Engineer of the Savannah, Florida and Western Railway (Plant System), Savannah, Ga., and, thereafter, until the end of 1886, he was employed as Designing Engineer for the Morse Bridge Company, at Youngstown, Ohio. In 1887 and 1888, he was Engineer with the Atlanta Bridge and Axle Works, Atlanta, Ga., and in 1888 and 1889, he was engaged as Principal Engineer with the King Bridge Company, at Cleveland, Ohio.

From 1890 to 1892, inclusive, Mr. Cowles served as Chief Engineer of the Youngstown Bridge Company, at Youngstown, Ohio. In 1893 and 1894, he was Assistant Manager of the Illinois Steel Company, North Works, Chicago, Ill., and from 1894 to 1898, he was Chief Engineer of the Pottsville Iron and Steel Company, at Pottsville, Pa. From 1900 to 1902, he was employed as Structural Engineer with the Brown Hoisting Machine Company, at Cleveland, Ohio, and from 1903 to 1910, he was Consulting Engineer and Secretary of the D. F. Holman Railway Track-Layer Company. In 1910 and 1911, Mr. Cowles served as Division Engineer, Division of Design, Chicago Passenger Subways, and, in 1912, he was appointed Structural Engineer of the Strauss Bascule Bridge Company. Since 1912, he had served as Structural Engineer of the Mead-Morrison Manufacturing Company, at Chicago.

His business life was that of a Civil Engineer of wide experience in different lines of work, and his life was dedicated primarily to his chosen profession and the happy home life which he enjoyed. In 1917 and 1918, Mr. Cowles took an active part in the Liberty Loan drives, Red Cross, and Y. M. C. A. work, as Secretary of his precinct. He was a member of the Illinois Chapter of the Society of the Sons of the American Revolution, and also of the Western Society of Civil Engineers.

* Memoir prepared by C. S. Churchill, M. Am. Soc. C. E.

His death, which was due to cancer of the stomach, occurred at his home in Oak Park, Ill., after an illness of nearly four months. The funeral services were conducted at the First Congregational Church of Oak Park of which Mr. Cowles was a member, and he was buried in Forest Home Cemetery.

He was married on February 11, 1891, in Beaver Falls, Pa., to Harriett J. Alford, who, with their daughter Marjorie B. Cowles, survives him. He is also survived by two brothers, Arthur C. Cowles, of Worcester, Mass., and Frederic M. Cowles, of Wallingford, Conn.

Mr. Cowles was elected a Member of the American Society of Civil Engineers on March 6, 1889.

IGNACIO MAREA de VARONA, M. Am. Soc. C. E.*

DIED MAY 12, 1922.

Ignacio Marea de Varona, the son of Ignacio M. and Angela M. (de Aguero) de Varona, was born at Puerto Principe, Cuba, on February 4, 1842. His ancestors were members of prominent Spanish families in Cuba. He received private instruction previous to entering Mr. Berger's French School, in New York City, where he was prepared for the Rensselaer Polytechnic Institute, Troy, N. Y. He entered the latter institution in September, 1860, and was graduated therefrom in 1863, with the degree of Civil Engineer.

Mr. de Varona began his professional work immediately after his graduation, and continued the practice of his profession throughout his long life. The earlier part of his career was spent on railroad work, in Cuba, South America, Europe, and the United States. From 1864 to 1869, he was Resident, Assistant, and Chief Engineer of the Nuevitas and Puerto Principe Railroad. As Chief Engineer, he designed and built the extension of that railroad to, and terminal works in, the Bay of Nuevitas. Later, from 1878 to 1881, he was again Chief Engineer of this railroad, and reconstructed important parts thereof.

In 1882, Mr. de Varona joined the Engineering Corps of the City Works Department of Brooklyn, N. Y., and was assigned to water supply work. This was the beginning of his connection with the development of the water supply of New York City, on which he was engaged until his retirement in 1914, after a service of more than thirty-two years. During the Eighties, he was assigned, first as Assistant Engineer, and, later, as Division Engineer, in connection with the extension of the water supply system of Brooklyn, east of Rockville Centre, N. Y. On the completion of this work, he was appointed, on July 1, 1894, by the Commissioner of Public Works, Mr. Alfred T. White, as Engineer of Water Supply, having charge of all the work connected with the extension and improvement of the water supply system, exclusive of the distribution system.

On the creation of Greater New York, in 1898, Mr. de Varona was continued as Engineer of Water Supply of the Brooklyn System, and, in 1902, he was appointed Chief Engineer of the Bureau of Water Supply, in charge of the Boroughs of Brooklyn and Queens, the distribution system at that time being also placed under his jurisdiction. He remained in charge of the Borough of Queens for only a short time, but continued as Chief Engineer in charge of Brooklyn until 1905, when he was appointed Chief Engineer with jurisdiction over all the Boroughs of Greater New York. In 1906, a separate Acting Chief Engineer was appointed for the Borough of Brooklyn, Mr. de Varona continuing in charge of the other Boroughs. In 1910, the organization of the Department was remodeled, and Mr. de Varona was appointed Chief Engineer of the Bureau of Water Supply, having charge of the maintenance,

* Memoir prepared by William W. Brush and Merritt H. Smith, Members, Am. Soc. C. E.

operation, and extension of the water supply system for the entire city, with the exception of the construction of the new Catskill System, which was under the jurisdiction of the Board of Water Supply, an entirely separate city organization. He remained in charge of the Bureau of Water Supply until July 1, 1914, when he retired on a pension. After his retirement, he continued the practice of his profession, as a Consulting Engineer, but was not very actively engaged on engineering work.

The important professional work of Mr. de Varona was the management and improvement of, first, the water supply system of Brooklyn, and, later, that of Greater New York. He was a man of unusual mental development and physical and nervous energy. As an engineering executive, he would quickly sense the strong and the weak points of any plan presented to him, or of any proposed line of action, and he energetically fought for what he thought represented the proper course. This was well exemplified in the proposed filtration of the Croton supply. In 1911-12, plans were prepared, under Mr. de Varona's direction, for a slow sand filtration plant, to be located on the site of the Jerome Park East Basin Reservoir, the total estimated cost of this work being about $8 000 000. After the bids for the main contract had been received, opposition to the work developed, the necessity for the purification of the Croton supply by filtration being questioned. No criticism was made as to the adequacy of the plans to accomplish the purpose sought, and Mr. de Varona strenuously fought for the filtration project, but the Board of Estimate ultimately decided not to proceed with this work.

The most important developments of the water supply system of New York City conducted under his direct charge, were: The utilization of the underground water supply of Long Island by wells and gallery systems; the design and construction of the high-pressure fire service systems in Brooklyn and in Manhattan; and the remodeling of the distribution system in Brooklyn. Mr. de Varona devoted virtually his entire time to the water supply system of New York City, occasionally, however, he aided other communities as Consulting Engineer, with advice regarding water supply and sewerage problems.

Although interested in the work of the Engineering Societies, Mr. de Varona did not frequently attend their sessions. Outside his official duties, his interest was especially in his home, where he died on May 12, 1922. He is survived by several sons and daughters.

Mr. de Varona was elected a Member of the American Society of Civil Engineers on April 7, 1886.

SOLOMON LE FEVRE DEYO, M. Am. Soc. C. E.*

DIED AUGUST 19, 1922.

Solomon Le Fevre Deyo, the son of Jonathan Nathaniel and Maria Le Fevre Deyo, was born in Gardner, N. Y., on December 17, 1850. He entered Union College at Schenectady, N. Y., and was graduated in the Class of 1870.

In 1870-71, Mr. Deyo was Instrumentman on the survey of the Town of Morrisania, N. Y., under the supervision of Gen. George S. Greene. In 1871, he entered the employ of the New York, New Haven and Hartford Railroad Company on the location of the Harlem River Branch, from Harlem River to New Rochelle, N. Y., and, later, was Resident Engineer on the construction of that road. In 1873, he entered the employ of the New York and Harlem Railroad Company, as Resident Engineer on the Fourth Avenue Improvement in New York City. In this position, he had charge of the section from Forty-seventh Street to Seventy-ninth Street, which work included the elimination of grade crossings and the construction of four tracks. He continued on this work until its completion in 1876.

From 1876 to 1881, Mr. Deyo was Superintendent of the American Metaline Company, New York City; from 1881 to 1883, Division Engineer on surveys with the South Pennsylvania Railroad Company; and from 1883 to 1885, Construction Engineer with the same Company. During 1886, he served as Principal Assistant Engineer on the Buffalo and Geneva Railroad, and, later, as Assistant Engineer with the Lehigh Valley Railroad Company.

In 1887, he was engaged in railroad reconnaissance in Northern Alabama for the Pioneer Mining and Manufacturing Company, and, later, in the same year, he was employed by the New York, New Haven and Hartford Railroad Company, as Assistant Engineer in charge of general work. He continued in this capacity until 1890, when he was appointed Assistant Engineer in charge of the Western District of the same road, resigning in March, 1900.

During the latter part of his service with the New York, New Haven, and Hartford Railroad Company, Mr. Deyo had supervision more particularly over the construction of additional tracks on the Harlem River Branch and the construction of the terminal yard at Oak Point, East River, including bulkhead, bridges, and approaches for the purpose of transfering cars to and from the boats.

In March, 1900, Mr. Deyo was appointed Chief Engineer of the Rapid Transit Subway Construction Company, New York City, which was organized by Messrs. August Belmont and John B. McDonald (after the latter had been awarded the contract) for the construction of the first underground rapid transit railroad in New York City. As is well known, the problems and difficulties which attended this pioneer line were many, and Mr. Deyo, who personally supervised the writing of the numerous specifications and agreements for the sub-contracts for the several sections of the work, by his tact, ability, and foresight, kept the work up to schedule and frequently acted as

* Memoir prepared by F. S. Curtis, Past-President, Am. Soc. C. E., and William J. Boucher, M. Am. Soc. C. E.

harmonizer between the Rapid Transit Commission, with its demands for progress, and the sub-contractors who were doing the work. So well did the organization function under his direction that the great work was opened to public use on October 27, 1904, just four years and seven months after the first official spadeful was turned. On January 10, 1905, a banquet was tendered Mr. Deyo by his employees and co-workers, at which approximately seventy, including guests, were present, and a large silver punch bowl was presented to him.

Mr. Deyo also held the position of Chief Engineer of the Subway Division of the Interborough Rapid Transit Company (the operating company) until July 1st, 1905. On the organization of the Interborough-Metropolitan Company in 1905, Mr. Deyo was appointed as Chief Engineer and held this position until 1918, when he retired from active work.

He was married on June 24, 1882, to Harriet Goodrich Brandon, who, with a daughter, Mrs. Royce R. Spring, survives him.

Mr. Deyo was of a quiet, reserved disposition, always approachable, having the confidence of all with whom he came in contact, not only those who worked for him, but also those for whom he worked. His fair treatment of all with whom he became acquainted was reflected in the number of his friends. His progress in life, in connection with his professional ability, was because of his kind and genial manner to everybody with whom he dealt.

He was a member of the Engineers' Club of New York City, which he had joined in 1893, continuing his membership during the remainder of his life. He had also been a member of the Holland Society of New York City since December 29, 1892.

Mr. Deyo was elected a Member of the American Society of Civil Engineers on June 6, 1888, and served as a Director of the Society from 1898 to 1900, and as Vice-President in 1904 and 1905.

WILLIAM CUSHING EDES, M. Am. Soc. C. E.*

DIED MAY 25, 1922.

William Cushing Edes, the son of Dr. Richard Edes, a noted Unitarian minister, and Mary Cushing Edes, was born in Bolton, Mass., on January 14, 1856. He was educated at the Massachusetts Institute of Technology, and was graduated therefrom in 1875 with the degree of Bachelor of Science in Civil Engineering.

Immediately after his graduation, Mr. Edes was engaged in statistical work for the United States Census Bureau. In 1877, he moved to San Francisco, Calif., and entered the service of the Spring Valley Water Company.

In 1878, he became connected with the Engineering Department of the Southern Pacific Company, and was employed as Assistant Engineer, until 1882, on the location of that Company's line through Arizona, New Mexico, and Texas. The experience gained during these years on a pioneer undertaking of such magnitude, and a natural aptitude for this class of engineering work, determined the course of his professional career, and practically the whole of his life after the completion of the survey was devoted to railroad location and construction.

In 1883, Mr. Edes returned to Boston, Mass., and was engaged in private practice until 1886, in which year he again went to California and re-entered the service of the Southern Pacific Company as Assistant Engineer, continuing with that Company until 1896. During this time, he was in charge of location and construction work for the Company, the most important of which was the location of the Coast Route in California, from San Luis Obispo to Ellwood, a distance of 107 miles.

In 1896, Mr. Edes was appointed Principal Assistant Engineer of the San Francisco and San Joaquin Valley Railway Company, which was building a line from San Francisco to Bakersfield, Calif., a distance of 381 miles. He was in immediate charge of both the location and construction of this line and personally made the location of that part of the road between Stockton and Richmond, Calif., 69 miles, the most difficult section. This road was later acquired by the Atchison, Topeka, and Santa Fé Railway Company and is now a part of its Coast Lines. In his capacity of Principal Assistant Engineer, Mr. Edes was thrown into contact with the people and the public officials of the districts traversed by the railroad, and his kindly personality, tact, and honesty contributed largely to overcoming the many difficulties encountered on projects of this kind and to facilitating the progress of the work.

In 1900, he returned to the Southern Pacific Company and was engaged, until 1905, on difficult location for the reconstruction of various parts of that Company's lines in California, Nevada, and Utah, chief among which was the section between Rocklin, at the western base of the Sierra Nevada Mountains, and Truckee, on the eastern slope. In 1905, he was appointed District Engineer of Maintenance of Way for the lines of the Southern Pacific

* Memoir prepared by the following Committee of the San Francisco Section: Bernard Benfield, Jerome Newman, and Frank Thompson Oakley, Members, Am. Soc. C. E.

Company in the northern part of California, filling that position for two years.

The Southern Pacific Company and the Atchison, Topeka, and Santa Fé Railway Company, in 1907, acquired jointly the Northwestern Pacific Railroad Company, which owned a line extending northerly from San Francisco Bay to Willits, Calif., 133 miles, and southerly from Humboldt Bay to Shively, Calif., 38 miles. It was decided to complete the gap between Willits and Shively, 106 miles, and Mr. Edes was selected as Chief Engineer to carry out the task of making the location and directing the construction of this link connecting the separated sections of the road. The line ran through the canyon of the Eel River, along precipitous banks of unstable material, and Mr. Edes' location through this country, one of the most difficult pieces of railway work on the Pacific Coast, is an enduring monument to his skill.

On the completion of this work, Mr. Edes was appointed by President Wilson as Chief Engineer and Chairman of the Alaskan Engineering Commission, which had been created to take charge of the location and construction of the Government railroads in Alaska. Immediately after his appointment, in May, 1914, Mr. Edes left for Alaska and, in April, 1915, sufficient progress had been made to recommend the adoption of the route by way of the Alaska Northern Railroad from Seward to Kern Creek, 71 miles, and thence by the construction of 400 miles of new road to Fairbanks. With the exception of timber suitable only for piling, ties, and small dimension lumber, there was nothing in the country with which to begin work; all construction material and equipment had to be brought in on steamers and facilities established for handling, storage, and repairs. The site of Anchorage, which was selected as the headquarters of the Commission, was practically uninhabited, and it was necessary to build a city at this place for the construction forces and towns at several other points for the development of the country and the administration of the innumerable activities connected with the railroad work. The country through which the road was to pass was an unbroken wilderness, entirely devoid of transportation facilities, and little was known about it. Notwithstanding these almost insuperable obstacles, at the end of 1915, there had been completed 35 miles of grading and 13 miles of track, as well as a considerable mileage of wagon roads.

In addition to the difficulties offered by the country, there was the added one of obtaining funds with which to prosecute the work vigorously. The United States was engaged in the World War, and Congress was not in a mood to furnish money for other than war purposes; it was necessary, therefore, for Mr. Edes to spend a large part of his time in Washington, D. C., in order to secure appropriations for continuing the work in Alaska. One of the Commissioners had been appointed Governor of the Territory, the other had re-entered the Army, and Mr. Edes was left as the only remaining member, in full charge and responsible for all the activities of the project. The duties were most arduous, and it is a high tribute to his honesty and ability that he succeeded in convincing Congress that funds for continuing the work should be provided, although the Nation was carrying on a war which taxed its power

and resources. The strain and worry under which he labored, however, had seriously affected his health, which was never robust, and as the general problems had been solved and the work carried well along toward completion, he resigned from the Commission in 1920, in order to take up work which would be less exacting. He was engaged after that time in consulting practice, advising especially on matters pertaining to railroad projects. It was while returning by train to San Francisco from an inspection of a project on which he had been consulted, that he was stricken with heart failure, resulting in his death.

Mr. Edes was a man who won the respect and affection of all those associated with him. In the words of one of his old friends:

"He was honest—with him honesty was an instinct. He thought along honest lines, and by honesty I refer not so much to money matters as to all the varied elements which occur in the life of an engineer. He was honest to those above him in giving them the benefit of his thought even though it might be at variance with the popular conclusions; he was honest with those working for him, and he was honest with himself in that he was careful to do what he thought was right.

"Mr. Edes was a modest man—if one might suggest it, too modest for his own good in this rushing, driving world. One would never learn from him of his achievements, and many were disposed to under-rate him because he was so self-effacing; but those who knew him well recognized the quality of the man and loved him for his gentleness and genuineness.

"The work he has done will live after him as a mark of his ability, and while, as time goes on, his name in connection therewith may be lost, yet the work itself will stand and be recognized as well designed and well done."

Mr. Edes was married on January 31, 1901, to Miss Mary Burnham, of Oakland, Calif., who, with three brothers, Robert, Francis and John Edes, and a sister, Mrs. Cyrus A. Roys, survives him.

He took an active part in public and charitable affairs, and was particularly interested in Red Cross work.

Mr. Edes was elected a Junior of the American Society of Civil Engineers on September 1, 1886, and a Member on November 4, 1896.

GEORGE RUSSELL FIELD, M. Am. Soc. C. E.*

DIED MAY 1, 1922.

George Russell Field, the son of Elisha and Martha Woodbury Field was born at Groton, N. Y., on October 1, 1871, and received his early education in the public schools of that place. He attended Cornell University, but was not graduated on account of severe illness.

In 1889, Mr. Field entered the Engineering Department of the Groton Bridge and Manufacturing Company, and remained with that firm for nearly five years. He served in the Drafting Room for a year and a half, making shop details and checking drawings. He was then placed in charge of design and contracting for bridge and structural work, and he also had charge of erection in the field. For about a year, he acted as Contracting Agent for the Bridge Company in New York, Pennsylvania, and New Jersey. In May, 1893, he was transferred to San Francisco, Calif., where he represented the Company as Contracting Agent until January, 1894.

On the latter date, Mr. Field became Contracting Engineer for the Risdon Iron and Locomotive Works in San Francisco and continued with that firm in that capacity until 1910. His position with the Risdon Company covered various works, among which may be mentioned the structural steel for the Ferry Depot, San Francisco, and for the Spreckels Sugar Refinery, at Salinas, Calif. His principal work, however, was in connection with the construction and installation of pressure pipe lines and gates for hydro-electric power plants, among which may be named the three plants of the Edison Electric Company of Los Angeles, Calif., the Northern California Power Company, the Puget Sound Power Company, and the Stanislaus Power Company of California.

In 1910, Mr. Field became Assistant General Manager of the Great Western Power Company, and served in that capacity until 1912. His duties with the Power Company were largely concerned with operation.

In 1912, he became interested in the Klamath River Packers' Association, and the following year he was made Vice-President and General Manager of the Company, a position which he held for nine years, until his death. In his capacity as General Manager, Mr. Field made a complete study of fish packing and re-organized the conduct of that industry for his Company.

His death on May 1, 1922, in Los Angeles, where he had gone for a visit, was due to an attack of influenza in 1919, from which he had never completely recovered. He was buried in Groton Rural Cemetery, at Groton, N. Y.

Mr. Field was married on January 26, 1905, in San Francisco, to Miss Gertrude White McCauley, who, with his father, Mr. Elisha Field, and a sister, Mrs. Carrie Field Rishford, both of Elmira, N. Y., survives him.

* Memoir prepared by J. D. Galloway, M. Am. Soc. C. E.

Mr. Field was a kindly, courteous gentleman, and to those who knew him well, the announcement of his sudden death brought a deep sense of sorrow and loss.

He was a member of the Bohemian Club of San Francisco and the Claremont Country Club of Oakland, Calif.

Mr. Field was elected a Member of the American Society of Civil Engineers on November 6, 1907.

CLEMENT ALEXANDER FINLEY FLAGLER, M. Am. Soc. C. E.*

DIED MAY 7, 1922.

Clement Alexander Finley Flagler was born at Augusta, Ga., on August 17, 1867. He was the son of the late Brig.-Gen. D. W. Flagler, Chief of Ordnance, U. S. Army, and his mother was the daughter of Gen. Clement A. Finley, Medical Corps, U. S. Army.

The traditions of his family and his inherited soldierly qualities led naturally to his choice of the Army as a profession, and after his graduation from Griswold College, Iowa, Mr. Flagler entered the United States Military Academy on June 14, 1885. He responded fully to the rigorous training of that institution and, from the first, became a prominent and popular figure in his class by reason of his exceptional mental ability, combined with the rare charm of his personality. Possessed of a remarkably fine physique, he was a leader not only in his studies, but in the military drills and athletic exercises, prescribed and voluntary, that formed an essential part of the West Point training. The horizon of the West Point cadet has broadened since his day. Then, except for the ten weeks' furlough at the end of the second year, one never saw beyond the rugged skyline of the surrounding hills from the day of his admission to the Academy to the day of his graduation, four years later. Whatever time was left for amusement and recreation from the crowded hours of study and duty was utilized, if at all, in such diversions as the limited facilities of the cadets could command. Cadet Flagler's energy, keen sense of humor, and originality of resource made him the leading spirit in these enterprises, and contributed to the great popularity he enjoyed among his fellows. The warm and intimate friendships which he formed at West Point continued throughout his life. He was graduated with high honors on June 12, 1889, and, on the same date, was assigned to the Corps of Engineers, U. S. Army.

His subsequent career was exceptionally varied and distinguished. After the completion of the three years' course of technical study at the Engineer School of Application at Willets Point, N. Y., to which the Engineer officers of the Army were assigned immediately after their graduation from West Point, Lieut. Flagler was stationed at San Francisco, Calif., as Assistant to the late Col. George H. Mendell, U. S. A., M. Am. Soc. C. E. for two years (1892-94), after which he was returned to the Military Academy as Instructor in Civil and Military Engineering. Thence, he was sent to Fort Monroe, Va., for a year's duty, and in 1896 to Portland, Ore., as Assistant in the Engineer District, with headquarters at that place. On the outbreak of the Spanish War, Capt. Flagler was promoted to the rank of Major and Chief Engineer of Volunteers, and served on the staff of Maj.-Gen. J. H. Wilson. He was in Porto Rico as Picket Officer and Assistant Engineer Officer, 1st Division, 1st Corps.

* Memoir prepared by Chester Harding, Brig.-Gen., U. S. A. (*Retired*), M. Am. Soc. C. E.

After the Spanish War, he was placed in charge of the Montgomery, Ala., Engineer District, for two years, and then was designated as Engineer Officer of the Department of Porto Rico and the Department of the East. He was in charge of the Wilmington, Del., Engineer District from 1904 to 1908, and of the Nashville and Chattanooga, Tenn., and Mobile, Ala., Districts for various periods from July, 1910, to July, 1913. While in the Wilmington District, he was a member of the Chesapeake and Delaware Canal Commission.

For the year ending May 7, 1914, Col. Flagler was a Student Officer at the Army War College, Washington, D. C. During the disturbances in Mexico, later in 1914, he served as Chief Engineer on the staff of the late Gen. Funston at Vera Cruz, and on his return from Mexico, he was detailed as Instructor at the Army War College until June 30, 1915.

At the time of the entrance of the United States into the World War, Col. Flagler was in charge of the Washington, D. C., Engineer District, including the water supply of the City of Washington, the Key Bridge across the Potomac, and the development of the Anacostia Flats. In August, 1917, however, he was sent to Fort Leavenworth, Kans., in command of the 7th Regiment of Engineers and the Engineer Officers' Training Camp. On December 17, 1917, he was commissioned Brigadier General, National Army, and was placed in command of the 5th Field Artillery Brigade, which he trained at Leon Springs and Waco, Tex., and which he took to France in May, 1918. Gen. Flagler commanded the Brigade at the front, in the St. Die, St. Mihiel, and Woevre Sectors. On October 17, 1918, he was promoted to the rank of Major General and served as Chief of Artillery, 3d Army Corps, in the Argonne-Meuse drive, from October 15 to November 20, 1918. After the Armistice, he commanded the 42d (Rainbow) Division in the march to the Rhine, from November 21 to December 14, 1918, when it became part of the Army of Occupation, with Gen. Flagler as Military Governor of Kreiss of Ahrweiler, which command he held until his return to the United States in March, 1919. On April 6, 1919, he was relieved of the command of the 42d Division, at Camp Bowie, Texas, and until April 6, 1920, was in command of the Camp and Engineer School, at Camp Humphreys, Va. His next duty was in Honolulu, Hawaii, as Department Engineer, Hawaiian Department, Commanding Officer of the 3d Engineers, and in charge of the Hawaiian Engineer District. From May 1, 1921, until his death on May 7, 1922, Gen. Flagler was at Baltimore, Md., as Division Engineer of the Eastern Division of Fortification and River and Harbor work under the Chief of Engineers, U. S. Army.

This record, derived from official sources, of active participation in the service of the Government, during periods of peace and war, for the thirty-three years between his graduation from the Military Academy and his death, emphasizes the confidence placed in Gen. Flagler by the authorities of the War Department. He was assigned to positions of high responsibility and authority for which his tried abilities had shown him to be qualified. He was by selection one of those to whom especial training was given for high command in time of war, and during the two wars that occurred within the scope

of his career, he held positions of great responsibility with the forces at the front. His energy, courage, and devotion to duty were inspiring to all those within his influence. Not only in the stress of battle, or in the performance of public duty in time of peace, but also in his intercourse with his fellow men, Gen. Flagler lived up to the highest ideals of honor and duty. The sense of devotion to duty, regardless of personal convenience or consequence, was never more highly developed than in him, and it constituted one of his most pronounced characteristics. He was a noble example of the high type of man who succeeds because he holds himself true to his honor, his duty, and his country.

Gen. Flagler was elected a Member of the American Society of Civil Engineers on March 13, 1917.

CYRUS GILDERSLEEVE FORCE, JR., M. Am. Soc. C. E.*

DIED FEBRUARY 7, 1922.

The death of Cyrus Gildersleeve Force, Jr., has removed another of those civil engineers whose activities left their impress on the Science of Engineering during a period when the development of municipal engineering work was in its infancy. The ranks of these pioneers, made up of a sturdy group of men, many of whom were without special educational training and all of whom, working with little precedent to guide them, really accomplished wonders, have been thinned by retirement or death until, to-day, only a few are to be found in active practice. It is impossible to estimate the importance of the work done by these men and its value to later engineers, but that it was of great importance is always realized, when, as death takes one of their number, one stops to consider and dwell on his accomplishments.

Cyrus Gildersleeve Force, Jr., who was a worthy member of this group of pioneer engineers, was born at Ledgewood, N. J., on August 27, 1841. He began his engineering career at about the end of the Civil War, having previously obtained his general education at the Chestnut Hill Academy at Succasunna, N. J., and at the State Normal School at Trenton, N. J., from each of which institutions he was graduated.

During several years, he taught school in New Jersey and Indiana, meanwhile preparing himself for the work which, later, became his profession. He early directed his studies into the field of bridge design and construction, and obtained employment with Zenas King, of Cleveland, Ohio, a pioneer in iron bridge construction and the founder of the King Bridge Company.

Mr. Force served as Chief Engineer for two years with this Company, during which time, many bridges in various parts of the United States were designed and completed under his supervision.

Prior to 1870, he entered the Engineering Department of the City of Cleveland, where he remained, in various capacities, for many years. The records show that he was City Engineer from 1884 to 1887, and, again, from 1890 to 1893, and, later, he returned to the Department as Assistant Chief Engineer from 1895 to 1899. Since 1899, he had devoted his time to consulting engineering practice in New Jersey, making his home at Ledgewood, where he died on February 7, 1922.

In August, 1892, Mr. Force was married to Miss Cora May Williams, of Cleveland, who, with a sister, survives him.

Mr. Force was greatly interested in bridge construction, and, during a part of his employment with the City of Cleveland, he specialized as a Bridge Engineer, and, later, as City Engineer, he retained a keen interest in bridge work. He served as a member of the Committee appointed to inspect the piers of the Brooklyn Suspension Bridge while it was being constructed.

During Mr. Force's time, the office of City Engineer was subject to change in personnel, depending on the political complexion of the Common Council.

* Memoir prepared by Robert Hoffmann, M. Am. Soc. C. E.

This explains the lapses in the continuity of his connections with the City Engineer's Office, and this condition of interrupted service makes it difficult to state definitely the exact work which may have been started by him or that which he continued or carried through to completion. Many projects started under his supervision were finished later under other supervision, and, likewise, work begun by others was carried on or completed by him. Under his direction, many miles of sewers and pavements were built in Cleveland. Mr. Force was an ardent advocate of the building of large sewers and substantial pavements. The large Walworth Sewer was one to which he gave much attention, both as to its design and construction. Several large bridges, one of which was the Central Viaduct, were under construction during his incumbencies of the City Engineer's Office, and received the benefit of his direction. Many other adaptions of engineering work which come under the direction of a city engineer, were efficiently considered and conducted by him, with the energy and the natural engineering ability for which he was noted.

The years which have elapsed since Mr. Force's engineering career in the City of Cleveland ceased, have proved his great worth and have shown him as a man whose life efforts must be conceded to have been of benefit to mankind.

Mr. Force was elected a Member of the American Society of Civil Engineers on February 6, 1878, and always took a keen interest in the welfare and work of the Society.

WILBUR FISK FOSTER, M. Am. Soc. C. E.*

DIED MARCH 26, 1922.

Wilbur Fisk Foster was born at Springfield, Mass., on April 13, 1834, being the third of six children. His father, Dexter Foster, of Ashfield, Mass., was a lineal descendant in the fifth generation of Peregrine White, who was born on the *Mayflower,* in November, 1620. His mother was Euphrasia Maria Allin, of Enfield, Conn.

After some years of preparatory studies, Dexter Foster began work in 1838 as Civil Engineer on the surveys, location, and construction of the Western Railroad of Massachusetts, from Springfield, Mass., westwardly to the New York State line, and of the Albany and West Stockbridge Railroad from the State line to Albany, N. Y. On the departure of Maj. George W. Whistler, the first Chief Engineer of the Western Railroad of Massachusetts, for Russia as Chief Engineer of the St. Petersburg and Moscow Railroad, Capt. John Childe became Chief Engineer, with Dexter Foster as his First Assistant, and he served as such until the completion of the road to Albany, N. Y., on April 22, 1843. Among the incidents of this service was the location and construction of the first railroad tunnel on a curve alignment. While engaged under Capt. Childe, on the survey of the Connecticut River Railroad, north from Springfield, Mass., Dexter Foster died on August 11, 1844.

The life of Wilbur Fisk Foster during the next seven years was a lively skirmish for his own support and such meager help as he could give his mother. His education was received in the public schools of Northampton and Springfield, Mass., and he was graduated from the High School at Springfield in 1851, with preparation for entry as a Sophomore at Yale University, but this was given up for lack of funds.

Capt. Childe who was then Chief Engineer of the Mobile and Ohio Railroad, gave him employment on that road. Mr. Foster left Springfield, on December 29, 1851, traveling by railroad to Cumberland, Md., thence by stage to Wheeling, Va., by steamboat to Paducah, Ky., and thence by steamboat to Hamburg, Tenn., near which place he joined the late Adna Anderson, M. Am. Soc. C. E., and a locating party, as Rodman. He was engaged on the location of the line to the mouth of the Ohio River and on preliminary branch lines to Columbus, Paducah, and Hickman, Ky., and Mt. Pleasant, Tenn. On the completion of the maps and profiles, the party was dispersed at Columbus, Ky. Mr. Foster went, as ordered by Capt. Childe, in company with Mr. Anderson, to Mobile, Ala., where he reported to Maj. John W. Goodwin, Resident Engineer. He was assigned as Instrumentman on the construction of the division from Boice to Quitman, Miss., in charge of the late Horace Harding, M. Am. Soc. C. E., Division Engineer.

On April 12, 1853, on order of Capt. Childe, Mr. Foster was relieved from duty on the Mobile and Ohio Railroad, with instructions to report to

* Memoir prepared by John Simpson Walker, M. Am. Soc. C. E.

Mr. Anderson at Columbia, Tenn., for service on the Tennessee and Alabama Railroad, of which Capt. Childe was Chief Engineer and Mr. Anderson, Resident Engineer. From this time until April 25, 1861, Mr. Foster was continuously engaged under Mr. Anderson who was Chief Engineer of the Central Southern Railroad, from Columbia, Tenn., south to the Alabama line, the Edgefield and Kentucky Railroad, and the Hendersonville and Nashville Railroad, which, together with the Tennessee and Alabama Railroad, when built, and after several consolidations, were finally absorbed into the Louisville and Nashville Railroad, constituting its line from the Alabama State line, at Veto, to Henderson, Ky.

Mr. Foster began work as Transitman on April 21, 1853, on the preliminary survey from Columbia, Tenn., toward Florence, Ala., and Hamburg, Tenn. On the completion of these surveys, he was assigned to duty as Instrumentman on the construction of the Second Division of the Tennessee and Alabama Railroad, from Spencer's Creek, north of Franklin, Tenn., to Spring Hill, under Mr. J. H. Devereux, Division Engineer, who afterward became a prominent and well known railroad executive. From July 24 to November 6, 1854, Mr. Foster had charge of track-laying from Nashville south, and, again, from June to November, 1855, he had charge of track-laying from Franklin south.

From November 5, 1854, to January 15, 1855, he was employed as Transitman on surveys of the Edgefield and Kentucky Railroad, from Nashville to the Kentucky line, in the direction of Trenton, Ky., and from January 15 to April, 1855, he served in the same capacity on the surveys of the Central Southern Railroad from Columbia, *via* Pulaski, to the Alabama State line, in the direction of Athens, Ala. The next few months were spent by Mr. Foster on a visit to his home in New England.

On the completion of his work on track-laying south of Franklin, Ky., he was engaged, on November 20, 1855, as Transitman on the surveys and final location of the Edgefield and Kentucky Railroad, from Edgefield Junction, Tenn., to Guthrie, Ky. During much of the time, he acted as Chief of Party. On the completion of the location and the usual plans and estimates, he was assigned to duty at the office in Nashville for the several railroad companies of which Mr. Anderson was Chief Engineer.

The first railroad bridge across the Cumberland River was that constructed at Nashville for the joint use of the Louisville and Nashville and the Edgefield and Kentucky Railroads. It was built, under the charter of the latter road, under the direction of Mr. Anderson, Chief Engineer. In September, 1856, Mr. Foster was assigned as Engineer in charge of this work, and made all the surveys, soundings, location of piers, and abutments, prepared all the plans, and supervised every detail until the completion of the masonry ready to receive the superstructure, which, combined with special duties from time to time on the different lines of the Company, was his work for two years. The masonry of the Cumberland River Bridge having been practically completed on August 1, 1858, he was given charge of the construction of the Third Division of the Central Southern Railroad from Pulaski to the Ala-

bama State line, at the present station, Veto, of the Louisville and Nashville Railroad. This construction included bridges over Richland Creek and Elk River, and a tunnel 1 230 ft. long, through Madry Hill, also much heavy rock grading and trestle work. On the completion of the tracklaying, he finished the work to a connection with the South and North Railroad at the Alabama State line on November 24, 1860.

From December 15, 1860, to April 25, 1861, Mr. Foster was employed on the surveys and location of the Hendersonville and Nashville Railroad, from Hopkinsville to Sebree, Ky., and in retracing the line and levels of the part already graded, from Guthrie to Hopkinsville and from Sebree to Henderson, practically completing the field work, maps, profiles, and estimates.

From the foregoing record, it is evident that Mr. Foster's work from the time of his arrival at Hamburg, Tenn., in 1852, was education in the highest degree, and his training therein had made him a competent civil engineer of experience.

He enlisted as a private in Company C (Rock City Guards), on April 25, 1861, and was mustered into service as a private in the First Tennessee Regiment, thus becoming a Confederate soldier. He was detailed on engineer duty in making surveys of the Tennessee and Cumberland Rivers near the Kentucky line, and had charge of construction at Fort Donelson, where he designed the water battery. Afterward, reporting to Col. Bushrod Johnson, Chief Engineer Officer, he was placed in charge of the construction of Fort Henry, although he remonstrated against its location. Mr. Foster was commissioned a First Lieutenant of Engineers on September 14, 1861, and ordered to Cumberland Gap, where he built earthworks, batteries, and block houses, and made a topographical map of the vicinity. When Gen. Kirby Smith advanced into Kentucky, Lieut. Foster was promoted to be Captain of Engineers, and was, successively, on the staffs of Generals Bragg and Joseph E. Johnston, in charge of topographical work. He was promoted to be Major of Engineers on March 17, 1864, and on August 7, 1864, he was assigned to the staff of Gen. A. P. Stewart, serving in that position until Johnson's surrender at Greensboro, N. C. In the performance of his duties, Maj. Foster was under fire in many battles, the most important of which were Chickamauga, Dalton, Resaca, New Hope Church, Kennesaw Mountain, Atlanta, Jonesboro, Franklin, and Nashville.

It has been stated that, "he was trusted implicitly by General Albert Sidney Johnston and General Joseph E. Johnston, and he was active in the Atlanta campaign under the latter, receiving high commendation for his work in map making and for the intelligence displayed in laying out the Confederate works, rifle-pits, breast-works, etc."

After the surrender, Maj. Foster returned with his detachment to Macon, Ga., where they separated, and he made his way to Nashville, walking part of the way. He reached Nashville on May 15, 1865, after four years' service in the Army of the Confederacy, which he stated was "an experience I shall never forget or regret." After a few days, he went to visit his mother in New England. He returned to Nashville on July 17, 1865, and shortly afterward opened

an office for the general practice of engineering, which he maintained until 1884, when it was merged with a contracting business, which soon became the principal feature. On July 15, 1865, the Broad Street Bridge Company appointed him as Engineer to supervise the reconstruction of the Suspension Bridge across the Cumberland River, at Nashville, which had been destroyed during the war. This work was completed, and the bridge was opened for traffic in August, 1866. On July 26, 1865, he was also appointed Engineer on the construction of the first street railway built in Nashville, and remained in charge of this work until its completion in 1866.

On January 1, 1866, Maj. Foster was made Resident Engineer of the Edgefield and Kentucky Railroad (now part of the Louisville and Nashville Railroad), and continued in that duty until November 15, 1869, during which period most of the bridges and trestles were rebuilt and the roadbed and track placed in good condition. From early in 1866 until July 1, 1872, when the Nashville and Decatur Railroad was leased to the Louisville and Nashville Railroad Company, he was constantly in the employ of J. W. Sloss, President of the Nashville and Decatur Railroad Company, as Engineer of the Company, and made frequent trips of inspection. Plans for stone arch culverts were prepared by him and were built under his direction. For many years, beginning in 1867, he was frequently employed at times by the Nashville and Cincinnati Railroad Company in the location of its real estate in Nashville and along the line; he also made an inspection and report on the condition of the bridges and trestles between Nashville and Hickman, Ky., in October, 1869.

From 1871 to 1876, Maj. Foster was entrusted with much engineering service for the Louisville and Nashville Railroad Company, about the most important being the location, acquisition of land, and plans for constructing a connecting line between that road and the Nashville and Decatur Railroad, in Nashville.

By the appointment of Mr. Sloss, President of the Nashville and Decatur Railroad Company, he made a preliminary survey in December, 1867, and furnished map profiles and estimates of cost of a proposed branch railroad from Culleoka to Lewisburg. In 1868, he made the necessary surveys and prepared a map of Davidson County, Tennessee, for the County Commissioners, at a cost of $3 106.

In 1869, he made a survey map, profiles, and estimates of quantities for a proposed railroad from Murfreesboro to Lebanon, Tenn., as well as preliminary surveys, map profiles, and estimates of quantities for a proposed railroad from Lebanon, *via* Hartsville, Tenn., to the State line in the direction of Scottsville, Ky. He also made a study of the State Fair grounds, cross-sectioned it, and designed and built a 1-mile trotting track, a grand stand, and a spur-track to the Nashville, Cincinnati, and St. Louis Railway. This is the same ground on which the Tennessee Centennial was held in 1897, and is embraced in the present Centennial Park. Laying out, grading, and macadamizing West End Avenue, from 15th Avenue to Cockrills Springs, and the construction of the Nashville and Fair Grounds Street Railroad from the city limits

to the Fair Grounds, was done by contract of the property owners, with Maj. Foster as Engineer and Contractor, the work being completed early in October, 1871.

The improvement of the Capitol grounds was undertaken by the Commissioners of the Penitentiary, in accordance with plans furnished by the late John Bogart, M. Am. Soc. C. E. Maj. Foster made a topographical map of the site and furnished a copy to Mr. Bogart who submitted designs. Maj. Foster took charge of the construction, developed the general and special designs, furnished all the detailed drawings of the cut-stone work, and had general supervision of the work in every detail. This work was done by convicts between July 12, 1871, and May 31, 1876, and no more pleasing retaining wall can be found anywhere.

On October 12, 1865, Maj. Foster was elected City Engineer of Nashville, and with the exception of the period from October 12, 1867, to July 1, 1869, he held the office by re-election or appointment until March 8, 1884, when he "peremptorily resigned". There were no records for guidance as to sewers, water pipe, street grades, no profiles, no field books, no record whatever of street alignment, or other details. There had been no control over improvements made by property owners, every one doing as it pleased him in the matter of grades, sidewalks, etc. When he resigned the office of City Engineer on March 8, 1884, Maj. Foster left the Department completely organized, and turned over to his successor complete records of more than three hundred streets, alleys, and sewers, as well as a thoroughly tested system of bench-marks and references covering the entire city at that date. The total cost to the city of the Engineering Department for the 16⅔ years of Maj. Foster's service, including every item of expense for salary, assistants, office rent, office expenses, etc., was less than $40 000. The work done during this period reflects the highest credit on Maj. Foster as a skilled engineer, possessed of high ideals and character, and a devoted public spirit. The City of Nashville was truly fortunate in having him for City Engineer during the time he served as such.

The North Nashville Bridge across the Cumberland River, near Hyde's Ferry, was built by the County of Davidson, represented by a Board of Commissioners, and under plans and specifications prepared by the late E. F. Falconnet, M. Am. Soc. C. E. The contracts for the various details were made by the Commissioners under the advice of Maj. Falconnet, and the work was begun and rapidly pushed under his direction. After his death on October 17, 1887, Mr. H. C. Gardner and O. H. Landreth, M. Am. Soc. C. E., were, successively, in charge. On February 28, 1888, Maj. Foster was appointed Engineer on the part of the Commissioners, and he directed the completion of the substructure and the inspection and erection of the superstructure until its completion and acceptance on April 1, 1889.

In April and May, 1891, Maj. Foster made surveys and prepared plans, profiles, and estimates for a boulevard, 80 ft. wide, from the Belmont School grounds southwardly 2½ miles, to an intersection with the Granny White Pike. Later, in 1893 and 1905, he made a subdivision of the Belmont Land Company,

north and east of this Boulevard. Of this work, Maj. Foster said: "This was my last specific engineering engagement, such other engineering service as rendered by me after 1884 being of little importance or incidental to my business as contractor."

The firm of Foster and Creighton was a partnership formed in 1885 by Maj. Foster and Mr. Robert C. Creighton, which was engaged in the contracting business. In 1907, Maj. Foster sold his interest in the firm and retired from business, and the Foster and Creighton Company was incorporated. Maj. Foster served as a Director in the Company for about five years, until 1912. The Company did a large amount of work, all of which was executed in a thorough, workmanlike manner, and was satisfactory to its patrons.

After Maj. Foster retired from business, he spent his time mostly in reading and writing. Most of his writing pertained to Masonry, but he prepared his engineering record, which furnished the data given herein. His war record was taken from the "Confederate Veteran" and "Confederate War History".

Maj. Foster was one of the charter members of the Engineering Association of the South, which was organized in 1889, and served as its President in 1899. The successor of this Association, the Nashville Engineering Association, made him an Honorary Member. He was also a Charter Member of the Hermitage Club and of the University Club, both of Nashville. At one time, he was a Director of the Phillips and Buttorff Manufacturing Company, and, at the time of his death, he was a Director of the Nashville Trust Company, which, on March 30, 1922, passed the following resolution:

"It is with profound sorrow we record the death of our dear friend and associate, Major W. F. Foster, who at 11:50 in the evening of March 26, 'passed from the land of shadow into the land of light'.

"Major Foster had served this Company on the Board of Directors, faithfully and efficiently, for twenty-five years.

"He was a man unusually endowed, of marked individuality, lofty ideals, broad sympathies, and unswerving integrity; an accomplished gentleman, a skilled civil engineer, a successful business man, an ardent Christian and churchman, and a distinguished Mason. His long life of eighty-eight years was a gospel of good will and good works, spent in unselfish devotion to God, his family and his fellow man. His influence was gentle, kindly and far-reaching. His presence always gave an added charm and a sweet old-fashioned dignity to any circle in which he moved.

"The memory of this good man is enshrined in our hearts, and we extend to his dear wife and loved ones, our deepest sympathy and commend them to Him 'who is able to ransom from the power of the grave and who turneth the shadow of death into the light of the morning'.

" 'When death draws down the curtain of the night, and those we love in his deep darkness hides, we can but wait the coming of the light, and bless the memory that still abides.'

"*Therefore, Be It Resolved,* That this expression of our respect and remembrance be inscribed on a memorial page of the Minute Book of this Board, and a copy be sent the family."

Maj. Foster joined the First Presbyterian Church of Nashville on September 8, 1868. He was elected a Deacon in 1873 and an Elder in 1896. He was

constant in attendance and a devoted member, performing all his duties with credit to himself and his church for more than fifty years. On his death, the pastor and officers of the First Presbyterian Church adopted the following resolution:

"On March 26, 1922, after a life of nearly four score and ten years, Major Wilbur Fisk Foster, having finished his course and kept the faith, was called from the church militant to the church triumphant, 'and entered into the rest prepared for the people of God.'

"Major Foster was a man of the strictest integrity and his character was unimpeached and unimpeachable. No one who knew him intimately could fail to admire his gentle and affable nature and to recognize in him a true man.

"In the death of Major Foster the city has lost a most valuable citizen, and one who has left his impress on it by his work as an official in past years, and his godly walk before the people.

"For nearly forty years he was an officer in this church and, until his physical affliction, in the loss of his hearing, prevented him from attending the meetings of the session, he was usually present, and his opinion and suggestions in regard to matters pertaining to the good of our church were always regarded as safe and sound.

"While we bow with submission to the will of our Heavenly Father, yet we cannot but mourn the great loss our church and official board have sustained in the death of Major Foster. We shall enshrine his memory in our hearts, and count ourselves happy to have had such a friend.

"*Resolved,* That a copy of these resolutions be sent to the family to whom we, as his Pastor and fellow officers, extend our sincere sympathy in their irreparable loss."

From the time he arrived at manhood, Maj. Foster gave so much of his time and attention to the Masonic order and its various branches, that his Masonic activities may be said to have formed a large and important part of his career. It is deemed appropriate, therefore, to set out briefly herein a record of his Masonic career, as authenticated by Mr. Garnett N. Morgan, Sovereign Grand Inspector General in the State of Tennessee, of the Supreme Council, Ancient and Accepted Scottish Rite, who states:

"It is very difficult to give in a very few words an adequate idea of the splendid Masonic career of Major Wilbur Fisk Foster 33°.

"His services to Freemasonry in Tennessee were brilliant in character and the honors of the Fraternity were showered upon him. He served not only as the Worshipful Master of the two Blue Lodges of which he was a member during his long and useful life, but became the presiding officer of each of the other Masonic Bodies in which he held his membership—Royal Arch Chapter, Council of Royal and Select Masters, and Commandery of Knights Templar. He was elected the presiding officer of the Grand Bodies of the State—Grand Lodge, Grand Chapter, Grand Council, and Grand Commandery—and served with distinction in each of these exalted stations. He also rendered eminent services to the General Grand Chapter of the United States.

"In the Ancient and Accepted Scottish Rite of Freemasonry, he assisted in organizing the Scottish Rite Bodies of the Valley of Nashville and for a number of years was Master of Kadosh of Trinity Consistory. In recognition of his services to the Rite, he was honored by the Supreme Council with the Thirty-third Degree.

"Major Foster was one of the most influential of Tennessee Masons, a Masonic scholar and author of note, and one of the ablest Masonic ritualists the State has produced."

Maj. Foster was married on June 19, 1866, to Miss Elizabeth Nichol, daughter of John Nichol, of Nashville, a member of one of the old and respected families of Tennessee. His widow, a daughter, Mrs. C. C. Foster, and two granddaughters, survive him.

Maj. Foster was endowed with unusual common sense, discretion, and a conscience. He was a modest, reticent, honest man, devoted to his family, his church and his Masonic connections, a gentleman of the old school, loyal to all interests confided to him, a thoroughly trained civil engineer, a brave, efficient Engineer Officer of the Confederate Army, methodical, prompt, and accurate. He enjoyed good health, and maintained all his faculties to the last, except that he was afflicted with deafness for several years before his death. He walked from his home to his business, and return, almost daily, until a few days before his death.

Major Foster was elected a Member of the American Society of Civil Engineers on May 7, 1873, and only a few living members of the Society have had a longer connection with it. Major Foster began his engineering work, the same year the Society was instituted.

SYLVAN EARL GANSER, M. Am. Soc. C. E.*

DIED APRIL 14, 1922.

Sylvan Earl Ganser, the son of Daniel Z. and Martha A. Ganser, was born at Harlan, Iowa, on December 21, 1879. He was graduated from the High School at Harlan in 1898, and attended Cornell College, at Mt. Vernon, Iowa, receiving his Bachelor of Science degree in 1904 and his Civil Engineer degree in 1906.

During the summer vacations of his college course, Mr. Ganser was employed as Surveying Assistant on the Illinois Central Railway and the Chicago and Northwestern Railway.

In 1905, he entered the service of the Union Pacific Railway Company as an Assistant Engineer on Maintenance of Way and was assigned to the Colorado Division, where he had supervision of minor improvements and maintenance work.

In February, 1907, he was employed by the Atlanta, Knoxville, and Northern Railway Company, a subsidiary of the Knoxville and Nashville, and, later, was transferred to second-track construction on the Louisville and Nashville Railroad as Resident Engineer. This work included heavy rock excavation, driving and lining of tunnels, and the enlargement of the Atlanta Terminals. Mr. Ganser was a member of a large organization engaged on the construction of new railroad lines, which, at that time, was in immediate charge of J. E. Willoughby, M. Am. Soc. C. E., Chief Engineer of Construction.

For one year he was employed by the Michigan Central Railway Company as Inspector on the placing of forms, concrete, and steel tubes for the Detroit River Tunnel, which connects Detroit, Mich., with Windsor, Ont., Canada.

In August, 1909, Mr. Ganser again entered the service of the Union Pacific Railway Company, as Assistant Engineer of Construction, in charge of the Pleasant Valley Line, on the North Port Extension, Denver Terminals, and the Fort Collins Line, and 250 miles of estimates and surveys, from North Platte, Nebr., to Ogden, Utah.

In 1911, he decided to specialize in structural steel design. He entered the Massachusetts Institute of Technology for additional technical training in September of that year and remained there for two years.

In June, 1913, he entered the service of the Duluth and Iron Range Railway Company, and made extensive studies for a gravity yard and a number of buildings and facilities for the Ore Terminal at Two Harbors, Minn.

From March, 1914, to September, 1915, Mr. Ganser was employed by the firm of Toltz, King and Day, as a Structural Engineer. In this capacity, he was responsible for, and contributed quite largely to, the success of the structural work on the Northern Pacific-Great Northern Office Building at St. Paul, Minn.

* Memoir prepared by H. E. Stevens, M. Am. Soc. C. E.

From September, 1915, to August, 1916, he was employed by the American Hoist and Derrick Company of St. Paul, and was responsible for the design of a large Government derrick for the Panama Canal and other machines of considerable importance.

In 1916, Mr. Ganser entered the service of the Northern Pacific Railway Company, with which he continued until his death on April 14, 1922. During this time, he contributed to the design of many structures, the most important of which was a reinforced concrete ore dock at Superior, Wis.

His death resulted from an operation, and was unexpected. His general health, while not of the best, had not interfered with his duties, and he was employed up to the time he entered the hospital.

Mr. Ganser was married in 1906 to Miss Bessie Stewart who, with his parents, survives him.

In character, his strongest traits were his uprightness, hatred of deceit, devotion to duty, love of home, hospitality, and charity. Mr. Ganser was a most kindly man, very humorous, and was deeply appreciated and greatly loved by his many friends. In all things, he was a man and a gentleman.

He was a member of the Official Board of Trinity Methodist Episcopal Church and a member of the Sunday School Board. He was also Vice-President of the Twin City Chapter of the American Association of Engineers.

Mr. Ganser was elected an Associate Member of the American Society of Civil Engineers on January 2, 1912, and a Member on January 18, 1921.

ROBERT CAMPBELL GEMMELL, M. Am. Soc. C. E.*

DIED OCTOBER 25, 1922.

Robert Campbell Gemmell, the son of Robert Brown Gemmell and Anna Eliza Campbell Gemmell, was born at Port Mathilda, Pa., on July 5, 1863. In the early Seventies, his parents moved to Topeka, Kans., where Mr. Gemmell secured a common school and high school education. Choosing Engineering as his profession, he entered on its study at the University of Michigan, from which institution he received his degree of Bachelor of Science, in 1884. In 1895, his Alma Mater conferred on him the degree of Civil Engineer, and, in June, 1913, in recognition of his valuable services as an Engineer, honored him with the degree of Master of Engineers.

Following his graduation, Mr. Gemmell entered the Engineering Department of the Atchison, Topeka and Santa Fé Railway Company, and became a Division Engineer thereof, in charge of construction work. In 1891, he located at Pendleton, Ore., and engaged in private practice, which extended over Oregon, Washington, and Idaho. In 1893, he moved to Salt Lake City, Utah, where, continuing his private practice, he was also employed in the Engineering Department of that city.

In the course of his practice in Salt Lake City, Mr. Gemmell was made Engineer for the mines of Capt. Joseph R. DeLamar, including the Golden Gate and the Consolidated Mercur prôperties at Mercur, Utah. He was the first State Engineer of Utah, holding that position from 1898 to 1901, when he went to Mexico as Manager of the Mexican Syndicate Mining Company, to become later Superintendent of the Mexican properties of the Guggenheim Exploration Company. Journeying to Spain in 1905, to examine some gold mining properties for a group of New York capitalists, he returned to Salt Lake City in January, 1906, and immediately became associated, in the capacity of Superintendent of Mine and Mills, with the Utah Copper Company, the banner copper mine of the world.

In 1909, Mr. Gemmell was appointed Assistant General Manager of the Utah Copper Company, and, on May 1, 1913, was made General Manager thereof. He was also Consulting Engineer for the Ray Consolidated Copper Company, the Chino Copper Company, and the Nevada Consolidated Copper Company, as well as General Manager of the Bingham and Garfield Railway Company. In June, 1919, he was advanced to the position of Assistant Managing Director of the Utah, Ray, Chino, and Nevada properties of the Utah Copper Company.

A lover of Nature, Mr. Gemmell was fond of outdoor sports, particularly of hunting, fishing, and golf. He freely gave of his life and means to all that promised betterment of American boyhood. Although his chosen profession was his life work, he devoted much of his time and talents to public affairs, and to numerous business enterprises wherein his conservative judgment and business sagacity were invited and appreciated. Gracious in manner, gentle in

* Memoir prepared by L. S. Cates, Gen. Mgr., Utah Copper Co., Salt Lake City, Utah.

spirit, manly in everything, his personality, not only commanded the admiration, but easily and quickly won the affection, of his associates, and gave him an acknowledged power among men. He was a true man whom it was a rare privilege to know and an honor to call friend.

As an engineer, Mr. Gemmell rose to eminence both in ideals and achievement. He built to the level of his best opportunities, because he built down to the solid foundation of a sense of obligation. Capable, thorough, skilled, and accomplished, he was patient and chivalrously considerate of those under his industrial authority to a surpassing degree, yet never satisfied with anything short of economic perfection. In his death, the Society has lost one of its most valued members, and the Profession one of its most able exponents.

He is survived by his widow, Belle Evans Anderson Gemmell, to whom he was married, at Salt Lake City, Utah, on October 17, 1888.

Mr. Gemmell was a Member of the American Institute of Mining and Metallurgical Engineers, the Mining and Metallurgical Society of America, and the Utah Chapter of the American Mining Congress (of which Local Chapter he was the prime organizer and first Governor). He was also a Member of the Sons of the American Revolution, and Honorary President and National Representative of the Salt Lake Council of the Boy Scouts of America. He was appointed by the Governor of Utah on the Committee which, in 1916, formulated the Workmen's Compensation Act of that State. He was a member of the Alta, University, Country, Commercial, and Rotary Clubs of Salt Lake City, of the Rudy Duck and the Bear River Duck Clubs, of Utah, the Flat Rock Club, of Idaho, and the Sequoyah Country Club, of Oakland, Calif.

Mr. Gemmell was elected an Associate Member of the American Society of Civil Engineers on October 5, 1892, and a Member on December 4, 1895.

RICHARD HENWOOD GILLESPIE, M. Am. Soc. C. E.*

DIED JULY 15, 1921.

Richard Henwood Gillespie was born in New York City on March 17, 1867, his parents having been William B. Gillespie and Elizabeth Henwood Gillespie. After completing his common school course, he entered Union College, Schenectady, N. Y., and was graduated with the degree of Civil Engineer.

From 1889 to 1891 Mr. Gillespie was engaged on railway reconnaissance, location, and construction in Tennessee, Virginia, and North Carolina, as Instrumentman, Topographer, and Resident Engineer. From 1891 to 1893 he was Engineer for the contractors on the Little Falls and Dolgeville Railroad, Little Falls, N. Y. In 1893 and 1894 he was Division Engineer on the location and construction of the Eastern Division of the Castagena-Magdalena Railroad, Colombia, South America. In 1894 and 1895 he was again Engineer for contractors, this time on Sections 10 and D of the Chicago Drainage Canal.

From 1896 to 1907 Mr. Gillespie was continuously in the service of the City of New York, as Transitman and Assistant Engineer in charge of construction in the Borough of the Bronx. From 1907 to 1910 he was Engineer for a contracting firm on the construction of a large sewer outlet in the Borough of the Bronx, and a section of the Catskill Aqueduct. During this engagement he designed a concrete form for sewers and aqueducts which was patented and adopted by a well-known firm of concrete form manufacturers.

From 1910 to 1918 Mr. Gillespie was Chief Engineer of Sewers and Highways, Borough of the Bronx, New York City, during which time from $2 000 000 to $3 000 000 was expended on highway and sewer work each year. In 1918 he took charge of the construction of sanitation work at Nitro, W. Va., for the smokeless powder plant there, and on January 1, 1919, he became Chief Engineer and General Manager of the Traylor-Dewey Construction Company, doing cement gun work principally. In the spring of 1921, he contracted a serious illness which resulted in his death on July 15, 1921.

Mr. Gillespie was married in 1896 and left a widow, Mrs. Hortense J. Gillespie, three sons, Harold, Richard, and Robert, and a daughter, Elizabeth.

His friends and associates sustained a great loss in his death, having appreciated his sterling character, engineering ability, and painstaking application to his professional work.

Mr. Gillespie was elected an Associate Member of the American Society of Civil Engineers on October 3, 1900, and a Member on June 5, 1906.

* Memoir prepared by R. A. MacGregor, M. Am. Soc. C. E.

CHARLES SEWALL GOWEN, M. Am. Soc. C. E.*

DIED OCTOBER 19, 1909.

Charles Sewall Gowen, the only child of James and Emeline (Cummings) Gowen, was born at Barnstable, N. H., on February 14, 1851. His father, who died in 1854, was descended from a Scotch-Irish ancestor who, with a large number of compatriots, came to America in the middle of the 18th Century. His mother was a descendant of Isaac Cummings, who came from Scotland in 1644 and settled in Ipswich, Mass.

Mr. Gowen received his early education in the public schools of Clinton, Mass., and was a student at the Massachusetts Institute of Technology from 1867 to 1869.

In October, 1869, he entered the service, as an Assistant Engineer, of the late James B. Francis, Past-President, Am. Soc. C. E., Agent and Engineer of the Locks and Canal Company, of Lowell, Mass., remaining there until May, 1876. Mr. Gowen's experience while with Mr. Francis was of great value to him in his later work.

In May, 1876, Mr. Gowen joined the forces engaged on the additional supply of the Boston Water-Works, serving until May, 1880, as Rodman, Leveler, and Assistant Engineer. Afterward, and until August, 1881, he was an Assistant Engineer in charge of the Western Division of the Boston Improved Sewerage, and, subsequently, he was made Resident Engineer and Superintendent of Construction of the Boston Water-Works, having charge of Dam and Basin "IV" of the Sudbury Supply near Ashland, Mass., until June, 1883. He then accepted an appointment as Assistant Engineer in charge of hydrography, with the Philadelphia Water Department, in investigations for a new source of supply from the Schuylkill River, and remained in this position until March, 1884, when he was appointed Assistant Engineer by the Aqueduct Commissioners of New York City, a body created by a law passed in the preceding year to obtain an additional water supply for the city.

During the summer and autumn of 1884, Mr. Gowen was engaged on surveys for reservoir sites and on the location of the "New" Aqueduct line from Croton Lake to the Harlem River, as well as on other studies in preparation for the building of this great Aqueduct, which, with the exception of a few short stretches of cut-and-cover and some short lengths of soft-ground tunnels, was in rock tunnel from the head-works at the Old Croton Dam to the gate-house at 135th Street and Convent Avenue, Manhattan, a total distance of 30¾ miles. The Aqueduct drops below the hydraulic gradient near the city line and continues under pressure a distance of 7 miles to the 135th Street Gate-house, reaching a maximum depth of 300 ft. below tide-water at the Harlem River Crossing, approximately 430 ft. below the gradient. There is also a short inverted siphon under "Gould's Swamp" near Tarrytown, N. Y. The remaining Aqueduct at grade has a horseshoe cross-section, 13.53 ft. high and 13.60 ft. wide at the springing line, and was designed for a capacity of

* Memoir prepared by Jules Breuchaud and Robert Ridgway, Members, Am. Soc. C. E., and Fred B. Rogers, Esq., New York City.

300 000 000 gal. per day, whereas the circular pressure tunnel below the city line was designed to carry 250 000 000 gal. daily. The conduit was brick-lined throughout. One of the details of the preliminary work was the accurate measuring of the line, which was done by methods developed by Mr. Gowen and Alfred Craven, M. Am. Soc. C. E. The measurements were made on the slope and were reduced to the horizontal after corrections were made for sag, temperature, and error in graduations. Bronze bolts were set on line in ledge rock or in granite monuments embedded in concrete at intervals of 1 000 ft. or more and were numbered and stationed. This work was done quickly and economically and reflected credit on Mr. Gowen and others who were responsible for the methods and the results. It proved its value during the construction period which followed. Mr. Gowen was an excellent surveyor and, when the tunnel driving was begun, established methods of carrying the lines and grades into the headings, which not only eliminated errors and insured accuracy, but also caused a minimum of interference with the contractor's operations.

When the Aqueduct work was placed under contract about the end of 1884, Mr. Gowen was made a Division Engineer and placed in charge of Division No. 1, which included the complicated gate-house and other head-works at the Old Croton Dam and Shafts 0 to 4, inclusive, of the Aqueduct. Shaft 0 was an inclined shaft, about 300 ft. long, and Shafts 1 to 4, inclusive, were vertical shafts, 250 to 350 ft. in depth. Later, a re-organization added Shafts 5 and 6 to Mr. Gowen's Division. Under his direction, the shafts were sunk and the tunnels from them were driven. A second re-organization in 1888 placed him in charge of the completion of the 13 miles of Aqueduct from Shafts 0 to 11, inclusive.

Soon after the masonry lining work was begun, much trouble was encountered, due to the lack of co-operation on the part of the contracting firm and the dishonest practices of some of its employees. The work was slighted in many ways, mortar was omitted, cavities were left behind the brick lining, and the work was greatly demoralized. Politics played its part, affecting seriously the morale of the inspection force. Mr. Gowen stood firm against these practices and, in spite of many attacks made on him by his opponents, insisted on having the defective work corrected. His resolute stand, in which he was supported by his subordinates as well as by the engineers of other divisions, led to an investigation of the work by the Legislature in 1888. This was followed by a re-organization of the Commission in the same year and the repair and reconstruction of the defective work at the expense of those responsible for it. The late Alphonse Fteley, Past-President, Am. Soc. C. E., who had been made Chief Engineer in 1889, stated in his report to the Aqueduct Commissioners covering the years 1887-95:

"Mention should be made, however, of the able services performed by * * * Division Engineer Charles S. Gowen * * *. Messrs. Rice, Gowen, and Craven especially have rendered to the City valuable service by successfully enforcing and superintending, under serious difficulties and hardships, the repairs of the defective parts of the Aqueduct."

Some of the difficulties encountered by the engineers in carrying out the work are shown in detail in the same report.

On the practical completion of the Aqueduct, Mr. Gowen, in 1890, was placed in charge of the construction of Reservoir "M" on the Titicus River, near Purdy Station, N. Y., one of the several reservoirs constructed at, and subsequent to, that time in the development of the Croton water-shed. The dam was a rubble masonry structure, faced with cut native stone, of a maximum height of about 135 ft., with earth wings, the latter containing rubble masonry cores. Its total length was 1 519 ft. The rolled earth embankment of the north wing was more than 100 ft. high and the whole structure was considered a work of considerable magnitude at that time. The dam impounded more than 7 000 000 000 gal., and the construction cost, with the roads and other appurtenances, was more than $1 000 000. Mr. Gowen gave his earnest attention to the organization of the job and established standards for quality of work which, with the rigid inspection of his subordinates and the co-operation of the contractor, resulted in an excellent structure.

The Titicus Dam was completed in 1895, but, in October, 1892, Mr. Gowen had relinquished the control of the work to take charge of the construction of the New Croton Dam, the contract for which was let in the same year. The Croton Dam is still one of the great dams of the world and, at the time it was designed, attracted the attention of hydraulic engineers everywhere. Its greatest height was about 300 ft., and the maximum depth of its foundations, below the river bed, was about 150 ft. One of its features is the curved spillway, 1 000 ft. long on the north side, the greater part of the length of which is parallel with the contours of the hill against which the dam abuts. The capacity of the reservoir is about 30 000 000 000 gal. The disintegrated limestone underlying the foundation at the deepest point gave much trouble, and the manner in which this feature of the work was treated is well described in a paper* by Mr. Gowen entitled "Foundations of the New Croton Dam". Before the dam was entirely finished, certain radical changes were made in the design of the south wing, calling for the destruction of the earth bank and core-wall which had been built there and the substitution of a masonry section. The changes were not approved by Mr. Gowen, and are the subject of his paper,† "The Changes at the New Croton Dam". This imposing engineering structure, with which Mr. Gowen's name is so closely identified, was finally completed by the end of 1906, together with the 32 miles of new highway about the reservoir. It constitutes a wonderful monument to him and to the contractors who built it.

Mr. Gowen did not wait for the completion of the Croton Dam, but on August 31, 1905, resigned from the service of the Aqueduct Commissioners and thereafter became engaged in consulting work. Until he was taken ill in 1909, he had hardly known a day's illness. In fact, his good health was a matter of comment among his associates. His illness continued for months until his death on October 19, 1909, at his home in Ossining, N. Y.

* *Transactions*, Am. Soc. C. E., Vol. XLIII (1900), p. 469.
† *Transactions*, Am. Soc. C. E., Vol. LVI (1906), p. 32.

Mr. Gowen had the spirit of a Spartan. Duty came first with him, and he could never understand why personal interests or convenience should stand in the way of a man's work. His reserve was sometimes misunderstood by those who knew him casually, and it was only on a closer acquaintance that one realized the more kindly aspects of his nature. There was little compromise in his make-up, and he had no patience with those who would slight their work for profit or for any other reason. The following is quoted from a memoir prepared by two of his friends and published in an Ossining paper soon after his death:

"A casual glance would give one the idea that Mr. Gowen was deeply reserved and self-absorbed, but an intimate acquaintance with him revealed the most charming qualities. He was schooled in strict integrity and was that most perfect creature of God's handiwork, 'an honest man'. His most pronounced characteristics were justice and fairness in his relations with those with whom he was associated. Political influence did not sway his action and in all things he was the soul of honesty and honor."

The same memoir contains this reference to his work at the Croton Dam:

"His administration of the work was characterized by the same rigid attention to detail which had marked his supervision of whatever he had previously undertaken. He zealously guarded the City's interests and was quick to detect and forestall any proposition that might lead to unnecessary expenditure. 'Now let us be severely logical' was a favorite expression in dealing with a perplexing problem. Eminently a man of the courage of his convictions, just and upright to an extreme, and the uncompromising foe of whatever looked like 'graft' and extravagance. A strict disciplinarian but, withal, a most loyal and steadfast friend to every man whose conduct justified his confidence. Indomitable energy was another strong characteristic. No degree of heat or cold, snow or blizzard, was sufficient to block the way, were it in the power of man to overcome the difficulties."

Mr. Gowen made many friends in and about Ossining, where he lived from 1885 until his death. He was interested in the community life and at the time of his death was President of the Briarcliff Golf Club, which office he had held since the organization of the club. He was also a member of the Shattemuc Yacht and Canoe Club of Ossining.

He contributed a number of papers and discussions to the *Transactions* of the Society, and, in addition to the two papers previously mentioned, he was the author of "The Effect of Temperature Changes on Masonry".*

He was married on June 1, 1882, to Miss Alice J. Fellows, the daughter of Mr. James K. Fellows, of Lowell, Mass. Mrs. Gowen, with two sons and two daughters, survived him.

Mr. Gowen was elected a Member of the American Society of Civil Engineers on March 7, 1888. He served as a Director from 1904 to 1906, and during that time was a member of the Finance Committee of the Board of Direction.

* *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 399.

HERBERT THOMAS GRANTHAM, M. Am. Soc. C. E.*

DIED NOVEMBER 4, 1922.

Herbert Thomas Grantham, son of Thomas and Anna (Bootes) Grantham, of England, was born at Athens, Pa., on September 15, 1868, and received his early education in the elementary and High Schools of his native town.

After his graduation from the High School, he immediately entered the employ of the Union Bridge Company, of Athens, Pa., one of the first companies in the United States to construct all-steel bridges. He remained in the service of that company several years, and then accepted more responsible duties with the Edge Moor Bridge Works, near Wilmington, Del.

Later, he was stationed in Philadelphia, Pa., where he was associated with some of the most prominent bridge and structural steel companies of the East, and in 1898 became Chief Engineer of the Belmont Iron Works. His energy, courage, and devotion to duty were inspiring to all with whom he came in contact, and his wise counsel added much to the prestige and growth of the company. In 1918 he was made its Vice-President.

He was known to his business associates as a man of sterling character, loyal to his friends, of pleasing personality, and excellent business judgment.

In the spring of 1893 he was married to Miss Alice Burton South, who, with a son, Leslie Burton Grantham, and a grandson, survives him.

Mr. Grantham was elected an Associate Member of the American Society of Civil Engineers on February 5, 1896, and a Member on May 7, 1902, and at his death held membership in the Union League of Philadelphia, Philadelphia Country Club, and the Aronimink Golf Club.

* Memoir prepared by George S. Webster, Past-President, Am. Soc. C. E.

HOMER HAMLIN, M. Am. Soc. C. E.*

DIED MAY 14, 1920.

Homer Hamlin was born on August 27, 1864, at Pine Island, Minn. He was educated in the public schools of Minnesota, and, later, was a teacher in the State.

In 1886, Mr. Hamlin went to San Diego, Calif., where he was employed for several years by the City Engineer. After leaving the City Engineer's office, he was engaged in private practice until 1894, when he left San Diego to make his home in Los Angeles.

From 1894 to 1901, he served in the offices of the County Surveyor of Los Angeles County and the City Engineer of Los Angeles. In January, 1899, he was appointed Chief Deputy of the Field Force in the latter office, a position which he filled for nearly three years. He left this position in 1901, to become an Engineer in the United States Reclamation Service, and while thus employed, he served as Project Engineer for the Yuma Project, which included the construction of the well known Laguna Dam across the Colorado River. He also made a study of the geological features of the Salinas Valley, in connection with a report which he prepared on the water resources of that section of California.

In August, 1906, Mr. Hamlin was appointed City Engineer of Los Angeles, and filled this position until July, 1917. He was also a member of the Advisory Board for the construction of the Los Angeles Aqueduct. During this period, a number of engineering works of notable character were constructed under his supervision. Particular mention should be made of the completion—under methods proposed by him and executed by City forces—of the Los Angeles Outfall Sewer. This work, which comprised a series of tunnels in water-bearing formation, was abandoned by the contractors, who declared that its completion was impossible. Other work done under his supervision included that at Los Angeles Harbor, the construction of the Hill Street Tunnel, the adoption, on his advice, of the Tilby rail, etc.

After leaving the employ of the City of Los Angeles, Mr. Hamlin engaged in private practice, devoting a large part of his time to work, as Consulting Engineer, for the Reclamation Service in Arizona, California, and Wyoming.

In connection with the Salt River Irrigation Project, and as a delegate representing the Association of Water Users of the Salt River Valley, he was attending a hearing before the Secretary of the Interior at Washington, D. C., when, on May 14, 1920, he was seized with a cerebral hemorrhage and died very suddenly while alone in his room at the Willard Hotel. He is survived by his widow and two children.

Mr. Hamlin was a distinguished member of the Engineering Profession in the Southwest, and a brilliant example of the type of man who is a real public benefactor. Modest, hardworking, conservative, and sound in judg-

* Memoir prepared by a Committee of the Los Angeles Section: J. B. Lippincott, M. Am. Soc. C. E., and the late Edgar True Wheeler, M. Am. Soc. C. E., with additional information supplied by W. T. Knowlton, M. Am. Soc. C. E.

ment, he has assisted materially in making the region of his adoption a place where a much greater number of people can reside happily. There is no nobler work that a man can do than to convert the desert into gardens, to improve the sanitation of cities, and add to the commercial development of the country by the construction of harbors, as Mr. Hamlin did. With all this, he was of a modest nature, shunning publicity, and avoiding ostentation over his accomplishments. His death is not only regretted by his friends, but is a distinct loss to the Reclamation Service and to the community in which he made his home.

Although Mr. Hamlin was largely a self-educated engineer, he was a natural student. In addition to his other technical studies, he specialized extensively in geological research. This was his recreation. His report on the "Water Resources of the Salinas Valley", written for the United States Geological Survey in 1901-02, was one of his most important geological studies. He was also identified with the early development of some of the oilfields of Southern California. A few months prior to his death, he had made a trip down the Colorado River from the mouth of the Virgin to Yuma, Ariz. This expedition was devoted particularly to the study of the possibilities of the great storage enterprise at the lower end of the Grand Canyon, at a point known as Boulder Canyon.

Mr. Hamlin was a member of the American Association of Engineers and of the Seismological Society of America. He was also a member of the Westlake Methodist Episcopal Church of Los Angeles.

The following resolution on his death was adopted by the Los Angeles Section of the Society of which he was a member:

"*Whereas,* It has pleased the Almighty to remove from our midst our fellow member, Homer Hamlin, this Society desires to express its sympathy to his bereaved widow and children, and to heartily condole with them in their poignant grief.

"His death will prove a distinct loss to his town, the State and the Nation, to whose service he has been, throughout his whole professional life, assiduously devoted.

"We desire to express our deep appreciation of his upright and manly character as an engineer and a citizen. As an engineer he was thorough, careful and reliable, honorable and conscientious, and not to be swerved from the straight path of rectitude by either fear or favor. As a citizen he was a model husband, father and neighbor, and his pure and moral life set a noble example for the young of all classes, and particularly for the members of his chosen profession.

"*Resolved,* That this resolution be spread upon the minutes of this Society, and a copy be sent to Mrs. Hamlin and family, with our deepest sympathy and condolence in their affliction.

" 'His life was gentle, and the elements so mixed in him that Nature might stand up and say to all the world, this was a man'."

Mr. Hamlin was elected a Member of the American Society of Civil Engineers on May 4, 1904.

WILLIAM AUGUSTUS HOWELL, M. Am. Soc. C. E.*

DIED MARCH 12, 1923.

William Augustus Howell, the son of George W. Howell and Mary Elizabeth (Ward) Howell, was born in Newark, N. J., on June 16, 1859. His father, who was of Welsh descent, was for many years City Treasurer of Newark. His mother was descended from one of the original founders of Newark.

Mr. Howell was graduated from New York University in 1878, receiving the degrees of Bachelor of Science and Civil Engineer. Three years later, he was awarded the degree of Master of Science.

He began his professional career in 1879 as Rodman on the Pennsylvania Railroad, rising by successive steps to Supervisor in 1889. A year later, he resigned his position with the Pennsylvania Railroad to become Chief Engineer of the New York, Susquehanna and Western Railroad.

In 1890 and 1891, Mr. Howell was Engineer of Construction of a ten-mile section of the Jamaica Railroad in the West Indies. A year later, he was in charge of the construction of an electric road between Newark and Jersey City, N. J., and, in 1893, he entered the employ of the City of Newark, where he remained until his death on March 12, 1923.

In the thirty years during which Mr. Howell served in the Engineering Department of Newark, he had charge of the grade elimination on the Pennsylvania, Delaware, Lackawanna and Western, and New Jersey Central Railroads, and for the past sixteen years he was the responsible head of the Street Paving Bureau of Newark.

In 1884, Mr. Howell was married to Katherine Brokaw MacDonald, of New Brunswick, N. J., who, with two married daughters and six grandchildren, survives him. Mr. Howell is also survived by a brother, Mr. George Coes Howell, who resides in California. His death came as a great shock to his many friends, as only three days before it occurred he had performed his usual duties in the City Hall.

He was a member of the American Society for Municipal Improvements, having served as President in 1915, and on many of its important committees, and was also the author of numerous reports and papers relating to street pavements. He was also a member of the American Road Builders' Association and the Newark Chapter of the American Association of Engineers.

Mr. Howell was a Republican in politics, and a member of the Presbyterian Church. He was a member of St. John's Lodge, No. 1, F. and A. M., the New Jersey Society, Sons of the American Revolution, the Delta Upsilon Fraternity, and Corinthian Council, Royal Arcanum.

Mr. Howell was elected an Associate Member of the American Society of Civil Engineers on April 3, 1907, and a Member on June 6, 1911.

* Memoir prepared by Morris R. Sherrerd and Edward S. Rankin, Members, Am. Soc. C. E.

TORAGORO KONDO, M. Am. Soc. C. E.*

DIED JULY 17, 1922.

Toragoro Kondo was born at Iino, a small village in the Niigata Prefecture, Japan, on June 1, 1865. His early education was acquired in the public schools of his native Province, where his aptitude for study was generally recognized. It was this ability that caused him to be chosen by the Government, to be sent to Tokyo, in order to complete his studies—no small honor to a country schoolboy. After two years of keen competition in the Yobimon, or Preparatory School, Mr. Kondo succeeded in entering the Imperial University at Tokyo, from the College of Engineering of which he was graduated with honor in 1887.

Immediately after his graduation, Mr. Kondo proceeded with his classmate, Mr. Kobayashi, to the United States to study the practice of engineering in the office of the writer at Kansas City, Mo., and he was also in the employ of other Western engineers. During that time, he obtained much valuable experience in the design and construction of railway and highway bridges, water-works, sewerage, and pumping engines.

Like most Japanese engineers, Mr. Kondo was an apt pupil and a keen observer; consequently, during his two-year sojourn in the United States, he learned far more than most recent American graduates of his age would have learned in twice that time. Not only was he of an exceedingly high order of intelligence, but he also possessed a personality that made him popular with everybody with whom he was associated. He was always kind, considerate, just, and honorable; and it was this combination of qualities, as much, perhaps, as his fine technical ability, that enabled him to attain the eminent and influential position he occupied in the Engineering Profession of Japan.

As Chief Engineer of the Home Department for a quarter of a century, he was charged with the approval of all the important engineering works undertaken in Japan, with the sole exception of those relating to the railroads.

In his earlier practice, he planned and superintended river improvements of importance; and, in 1896, the Ministry of Education conferred on him the honorary degree of Kogaku-Hakushi, which corresponds both to the French degree of Docteur des Sciences Appliquées and to the American degree of Doctor of Engineering. There is no distinction greater than this in the Japanese Empire; for, like membership in the Académie des Sciences of France, it stamps a man as eminent in science, and, in addition, it indicates that he is a successful technician. The selectness of this degree is most jealously safeguarded by the Government, and it is never bestowed for any other reason than real worth. In order to take this degree, Dr. Kondo presented an elaborate thesis on Hydraulics, in which field he had been studying for nearly a decade.

He was for years a member of the Earthquake Investigation Committee and also of the Harbor Commission. In every large exposition held in Tokyo

* Memoir prepared by J. A. L. Waddell, M. Am. Soc. C. E.

since 1907, he had served as one of the Jurors. In 1915, Dr. Kondo accepted a chair in the Imperial University of Tokyo, and after that date, he divided his time between administrative and academic work.

A brother engineer, who, to-day, stands at the head of the Profession in Japan, I. Hiroi, M. Am. Soc. C. E., writes, as follows:

"Dr. Kondo, besides being an engineer of remarkable ability, was a man of the highest integrity. In the performance of his duties the most important one of which was the examination of plans submitted for approval by resident as well as prefectural engineers, and by individuals for permits from the Ministry, he was never influenced by either fear or favor. His professional life was one of unswerving rectitude and vigilant discipline. He had far more lucrative positions offered to him from time to time by various corporations and individuals; but he faithfully stuck to his post until the day of his death. His fine character and affable nature won for him many friends and made him both respected and liked by all those with whom he came in contact. To young engineers he was particularly friendly, and he helped many of them to rise in the profession."

Dr. Kondo was married in 1901 to Miss Koh Kato, the daughter of an eminent Japanese philosopher and statesman. He is survived by his wife, three sons, and a daughter.

Dr. Kondo was elected a Junior of the American Society of Civil Engineers on October 3, 1888, and a Member on June 5, 1901.

EMIL EDWARD KUERSTEINER, M. Am. Soc. C. E.*

DIED MARCH 10, 1913.

Emil Edward Kuersteiner was born on March 26, 1857, at New Orleans, La., where he resided until his parents removed to Cincinnati, Ohio, in 1878. He was graduated from the Woodward High School at Cincinnati in 1874, afterward going to Europe to obtain his engineering education. He was graduated from the Polytechnic School of Switzerland, at Zurich, in 1879, with the degree of Civil Engineer, and remained at his Alma Mater as Assistant to Professor Culman, Professor of Graphical Statics and Civil Engineering, for a year, returning to the United States in the Fall of 1880.

From 1881 to 1885, Mr. Kuersteiner was employed as Draftsman by the Cincinnati Southern Railway and the New Orleans and Northeastern Railway, under the late L. G. F. Bouscaren, M. Am. Soc. C. E.; and from 1885 to 1888, he served as Draftsman and Computer with the Louisville Bridge and Iron Company of Louisville, Ky., with which Company he remained, in charge of the Drawing Room, until 1896. In 1896, he removed to Johnstown, Pa., and took charge of the Drawing Room of the Johnstown Company; but in 1898, he returned to Louisville, and was engaged in the Bridge Department of the Louisville and Nashville Railroad Company, "on trial for a period of thirty days."

W. H. Courtenay, M. Am. Soc. C. E., Chief Engineer of the Louisville and Nashville Railroad, writes of Mr. Kuersteiner, as follows:

"He acted in the capacity of Assistant Bridge Engineer until November, 1906, when he became Bridge Engineer, and served in that capacity until a short time before his death. His work covered a long period of reconstruction of bridges for heavier loads on operated lines, and the construction of numerous structures on new lines. Until 1904, he and the Bridge Engineer practically handled all the bridgework, preparing plans for the masonry and checking the designs for the steelwork. All this work was very exacting, but he performed his work with apparent ease, as he was an excellent mathematician, an able and accomplished engineer, and a very accurate and tireless worker. He had a host of friends, all of whom knew him as Mr. 'K'. He was a man to whom young men went for advice and help. His opinions in matters pertaining to his profession and community interests were often sought by others and were always found to be exceedingly helpful and valuable, as he was a cultured, educated man of clear judgment."

Mr. Kuersteiner was a man who sought usefulness rather than preferment. He always helped others when it was within his power, without first thinking of his own interests, and he was intensely loyal to his work, his friends, and his ideals.

He was married in Louisville, Ky., and is survived by his wife, four sons, and a daughter.

Mr. Kuersteiner was elected a Member of the American Society of Civil Engineers on December 1, 1897.

* Memoir prepared by William Mayo Venable, M. Am. Soc. C. E.

DAVID MERIWETHER, JR., M. Am. Soc. C. E.*

DIED JUNE 20, 1922.

David Meriwether, Jr., was born at Louisville, Ky., on February 27, 1879. He was graduated from the Rose Polytechnic Institute, in the Class of 1900.

Immediately after his graduation, Mr. Meriwether entered the service of the Pennsylvania Railroad Company, in the Engineering Department. In 1901, he secured a position in the Construction Department of the Southern Railway Company, in which position his attention was given to location and construction work. He remained with this Company until March, 1920, during which period he rose from the position of Instrumentman to Assistant to the Chief Engineer. In addition to directing construction work on many miles of second track, he designed many yard layouts and terminals and some of the most important concrete structures on the Southern Railway.

In March, 1920, Mr. Meriwether entered the service of the United States Railroad Administration as Regional Engineer in charge of the Allegheny Region. In this capacity, he was called on to solve some of the difficult problems resulting from Federal control of the railroads, a task for which he was eminently fitted by his experience and judicial temperament. He was engaged in this work at the time of his death on June 20, 1922. He is survived by a widow and two sons.

Mr. Meriwether was a thorough and painstaking engineer. During the last twenty years of his life, he was closely associated with the writer, and his sterling integrity endeared him to all with whom he came in contact.

Mr. Meriwether was elected a Member of the American Society of Civil Engineers on November 9, 1920.

* Memoir prepared by E. M. Durham, Jr., M. Am. Soc. C. E.

MACE MOULTON, M. Am. Soc. C. E.*

DIED APRIL 27, 1909.

Mace Moulton was born in Manchester, N. H., on February 15, 1855, of a distinguished family, his father being a man of great scientific attainments, and his grandfather, whose name he bore, at one time a Congressman from New Hampshire. In 1872, Mr. Moulton entered the preparatory department then maintained by the Thayer School of Civil Engineering, on the discontinuance of which his preparatory studies were made in the Chandler Scientific Department. His studies were interrupted from time to time when he engaged in professional work.

After his graduation, and for six months, until March, 1870, he was Assistant Engineer in the Department of Maintenance of Way of the Eastern Railroad, at Salem, Mass. Then, for most of the time during the next four years, he was associated with the late C. Shaler Smith, M. Am. Soc. C. E., of St. Louis, Mo., the noted bridge engineer, who designed and built the first cantilever bridge in the United States. During this period Mr. Moulton designed numerous important railroad bridges.

From June, 1883, to December, 1884, he served as Principal Assistant Engineer to the Edge Moor Iron Works, Wilmington, Del. In 1885 he was Chief Assistant in the construction of the Kentucky and Indiana Bridge over the Ohio River at Louisville. In 1886-87 he was for two years Engineer of Bridges of the Colorado and Midland Railway, designing and superintending the construction of all the bridges and buildings of the road between Colorado Springs and Leadville, Colo. For a year and a half, in 1888-89, he was in private practice in Boston. In 1889-90, he was for two years Consulting Engineer of the Berlin Iron Bridge Company, of East Berlin, Conn.

From 1891 to 1896 Mr. Moulton was Chief Engineer of the R. F. Hawkins Iron Works, at Springfield, Mass., building many important railway and highway bridges and many iron and steel frames for buildings. From 1896 to 1903 he was engaged in private practice in Springfield, designing and constructing many steel structures throughout New England, and acting in a consulting capacity for various cities and railroads.

Since 1903 Mr. Moulton was a Consulting Engineer in New York City. Among his tasks was the reconstruction of the bridge across the Hudson River, at Poughkeepsie, an exceedingly difficult and complicated piece of engineering, which was accomplished successfully, the expenditure, nearly $1 500 000, being kept within the original estimate. In 1907 he was made President and Chief Engineer of the Milbrook Company and its Allied Companies, capitalized at $20 000 000, and aiming to provide rapid transit for the northern part of New York City and the neighboring parts of Westchester County. This position he held at the time of his death.

Mr. Moulton's first wife, Emma Blaisdell, of Hanover, N. H., died during their residence in Springfield, and, later, he married her sister who survived

* Memoir compiled by Robert Fletcher, M. Am. Soc. C. E., the first part from an obituary in the *Dartmouth Bi-Monthly* for June, 1909; the second part from correspondence and data in the office of the Thayer School of Civil Engineering.

him. There were three children, Mace, Jr., who was associated with his father in business, Thornton, a student at the University of Pennsylvania, and Mrs. Walter O. Greene, of Wakefield, N. J.

It is appropriate to add a few facts concerning the earlier years of Mr. Moulton's career: When a student he often surprised the writer by a certain degree of precocity (he looked younger than he was) when he betrayed unexpected aptitude in his grasp of engineering principles and details of practice. He was a very neat draftsman and penman, accurate in computation, and had a fine artistic sense. When the office procured one of Edison's "electric pens" (the precursor of the mimeograph) for manifolding purposes, he designed and printed an ornate letter head which showed his artistic skill with an entirely new implement.

Mr. Moulton's graduating thesis, on the Haverhill (Mass.) Drawbridge, then just finished, was a model in completeness and execution. With few suggestions from his instructor, he made the measurements by himself on the spot, all the elaborate calculations, full small-scale drawings showing many of the details, and wrote a clear description of the work. This unusually complete description and careful analysis was presented in about 120 manuscript pages and more than 25 diagrams and colored drawings. Undoubtedly, this was the means of his receiving, about a year later, an offer from Mr. C. Shaler Smith, to take him as an Assistant. During his course of training under this distinguished bridge designer, of whom he became an apt and devoted pupil, Mr. Moulton occasionally communicated with the writer telling of his work, and sent interesting notes, sketches, and descriptions exhibiting Mr. Smith's methods and several novel and original features of designs from the master mind. Among these was Mr. Smith's novel device for hanging the railroad in the Royal Gorge of the Arkansas River from iron rafters spanning the narrow stream; also Mr. Smith's predilection for the use of the continuous girder in certain cases; and his ingenious use of a link in the end panel of double-intersection trusses to distribute automatically the stress in the two systems so as to diminish the ambiguity. He told the writer afterward that Mr. Smith "believed in young men." Certainly that was so in this case, for Mr. Moulton was entrusted with the entire design of the Sabula Drawbridge, under the governing conditions imposed by Mr. Smith. The notes and blueprints relating to this work constituted an admirable example of the very best practice in bridge design at that date.

It was during the student days of Mr. Moulton that the method of graphic statics was brought to the notice of the Profession by the late Professor A. J. DuBois, M. Am. Soc. C. E., of Yale, through *Van Nostrand's Engineering Magazine*. Mr. Moulton at once became an enthusiastic disciple in mastering the principles.

Just before his death, Mr. Moulton was elected President of the Thayer Society of Engineers, of Dartmouth College. In later years, he gave the impression of being quick to assert the rights, dignities, and standing of his Profession, an example which, if more generally followed, would enhance the general and public recognition which should be accorded to it.

Mr. Moulton was elected a Member of the American Society of Civil Engineers on June 4, 1884.

ROBERT MORRIS NEWMAN, M. Am. Soc. C. E.*

DIED NOVEMBER 13, 1922.

Robert Morris Newman, the son of Edward and Eliza Newman, was born in Cheltenham, England, on August 1, 1839. He was educated at the University of Montreal, Canada, and on January 8, 1859, was articled for three years to Joseph William Burke, a practicing Civil Engineer and Provincial Land Surveyor, of Canada. He practiced afterward as Civil Engineer and Surveyor until July, 1865.

From July to November, 1865, Mr. Newman was Assistant City Engineer of the City of Fort Wayne, Ind. From December, 1865, until October, 1868, he served as Assistant Engineer on the Pittsburgh, Fort Wayne and Chicago Railroad. From October, 1868, until April, 1870, he was Resident Engineer on the Massillon and Cleveland Railroad, in charge during the location and construction of 13 miles of road. From April, 1870, until February, 1871, he was Resident Engineer of the Michigan Air Line Railroad (now the Air Line Division of the Michigan Central Railroad) during its construction between Jackson and Niles, Mich., a distance of 103 miles.

From February, 1871, until August, 1873, Mr. Newman was a Resident Engineer and then Principal Assistant Engineer for the Michigan Central Railroad, being stationed at Jackson, Lansing, and at Cheboygan, Mich. During this period the present passenger station at Jackson was built under his supervision, also locomotive shops and houses, water stations, second tracks, etc. The late Henry A. Gardner, M. Am. Soc. C. E., was then the Chief Engineer of the Michigan Central Railroad.

From August, 1873, to some time in 1875, Mr. Newman was Assistant Engineer on the Erie Railway. He then returned to Jackson, where he remained until about 1880 and where he was married to Miss Kate Smith, daughter of Hiram H. Smith. During this period of his residence in Jackson, he was engaged more or less as an Assistant or Resident Engineer with the Michigan Central Railroad.

During 1880 and 1881, Mr. Newman was Assistant Chief Engineer with the Northern Pacific Railroad, being stationed at Jamestown, S. Dak., and Glendive, Mont. In 1881, he went to Minneapolis, Minn., and became interested in the manufacture of newsprint paper, the firm name being Warner, Newman and Elfelt. He remained in Minneapolis until 1887, and then returned to Jackson to become City Engineer, taking office on April 16, 1888. This position he retained until 1902.

From 1902 until 1910, Mr. Newman was Manager of the Jackson Vehicle Company, retiring when this concern was reorganized as the Jackson Automobile Company. He was a Director of the Longyear and Mesaba Land and Iron Company of Minnesota, from about 1895, and made occasional trips to this iron region.

* Memoir prepared by William G. Fargo, M. Am. Soc. C. E.

Mr. Newman served from 1903 to 1910 as Vestryman of St. Paul's Protestant Episcopal Church, of Jackson, and was a member of the principal local clubs. He was a dignified and courtly gentleman and a capable business man, as well as a thorough engineer-executive possessed of excellent judgment.

Mr. Newman was elected a Member of the American Society of Civil Engineers on May 6, 1874.

FRANK CHITTENDEN OSBORN, M. Am. Soc. C. E.*

DIED JANUARY 31, 1922.

Frank Chittenden Osborn, the son of Reuben Howard and Livonia (Chittenden) Osborn, was born at Greenland, Mich., on December 18, 1857. His early education was acquired at Calumet, Mich., and he received his technical education at Rensselaer Polytechnic Institute, Troy, N. Y., from which he was graduated in Civil Engineering in 1880.

After his graduation, Mr. Osborn entered the employ of the Louisville Bridge and Iron Company as Assistant Engineer, which position he held until 1885, when he became Principal Assistant Engineer of the Keystone Bridge Company. In 1887, he joined the firm of G. W. G. Ferris and Company, Inspectors and Designers of Structural Steel Work, with headquarters in Pittsburgh, Pa.

In 1889, he accepted the position of Chief Engineer of the King Bridge Company, of Cleveland, Ohio, but resigned in 1892 to open an office for the private practice of engineering. This business developed to such an extent that, by 1900, it was incorporated as The Osborn Engineering Company, and became one of the best known firms of the Middle West. Mr. Osborn was connected with the Company as Director at the time of his sudden death on January 31, 1922, at his home in Cleveland, Ohio.

Mr. Osborn's professional life was devoted to the fabrication of structural steel in bridges and buildings. His Company designed a large number of steel and reinforced concrete bridges for cities and corporations throughout the United States, including those for steam and electric railways and grade eliminations. These bridges included two-hinge and three-hinge arch types, spandrel-braced, and plate-girder construction, and among them was the Y-bridge, a three-arm structure over the Licking and Muskingum Rivers, at Zanesville, Ohio, at that time the largest reinforced concrete bridge in the United States.

Mr. Osborn's keen interest in good designs for bridges led him to make a collection of views of many of the best and most beautiful of such structures throughout the world. He was a pioneer in concrete and reinforced concrete construction, as has been shown, and his studies extended to the design and construction of nine Portland cement plants, with a producing capacity of 17 000 bbl. of cement per day.

In 1908, Mr. Osborn was appointed a member of the Cuyahoga County Building Commission, which Commission had charge of the erection of the new Court House in Cleveland. This is one of the most notable buildings in that part of the State and cost $5 000 000. He was also a Director of the Lake Shore Banking and Trust (Savings) Company, in Cleveland.

He was the author of a valuable treatise entitled "Tables of Moments of Inertia and Squares of Radii of Gyration", which was published in 1886. He also devised a code of conventional signs for bridge riveting, which is now in general use, and proposed to bridge engineers and to manufacturers a

* Memoir prepared by Willard Beahan, M. Am. Soc. C. E.

standard size of sheet, of 24 by 36 in., for bridge drawings, which is also in general use.

He was one of the early (1889) members of the Cleveland Engineering Society, having served as its Secretary in 1897, and its President in 1898. He was a consistent contributor to its publications and presented many books to its Library, served on its important committees, and was most active in its affairs to the time of his death. He was also a member of the American Railway Engineering Association, the American Society for Testing Materials, and the Institution of Civil Engineers of Great Britain.

Mr. Osborn was a member of the University Club and of the Athletic Club of Cleveland, and was Chairman of the Advisory Board of the Cleveland Technical High Schools. He served as Vice-President of the Alumni Association of Rensselaer Polytechnic Institute in 1910, and was earnest in his relations to his Alma Mater, lecturing to the Senior Class, and, in 1910, he delivered the address before the Graduating Class. He was a member of the Sigma Xi Society, an honor society of technical graduates chosen by the Alumni of his Alma Mater.

He was very active in civic and patriotic affairs, his interest having been prompted by his lineage. He was a member of the Society of Colonial Wars, the Sons of the American Revolution, the New England Society of Cleveland, the Western Reserve, the Western Reserve Historical Society, the Navy League, the Aerial League of America, and the National Geographic Society.

Mr. Osborn was also a member of the Masonic Club, and a very active and earnest Mason, his membership in Masonic Fraternities dating from 1891. He was a member of Tyrian Lodge, Cleveland Chapter, Cleveland Council, Holyrood Commandery, Lake Erie Consistory, and Al Koran Temple.

As an engineer, Mr. Osborn was a pioneer in the best class of bridge and structural design, and he has left much excellent construction showing originality and early study. As a citizen, he gave unselfishly of his best efforts and much time and strength to those things which insure the safety, health, and happiness, of his fellows.

As a man, he was most kindly to the younger men of his profession, and gladly welcomed the engineer who was a stranger to his city. His was a service rendered beyond remuneration, and he made the interests of his clients and his neighbors his own.

On October 27, 1880, Mr. Osborn was married at Calumet, Mich., to Annie, daughter of Stephen and Amelia Hodgess Paull. His wife and their son, Kenneth Howard Osborn, M. Am. Soc. C. E., survive him.

Mr. Osborn was elected a Member of the American Society of Civil Engineers on October 3, 1888. He served as a Director in 1901-03, and was a member of the Special Committee on Steel Columns from 1909 to 1916. He had been a member of the Cleveland Section from its organization, and had served on its Committee on the Cuyahoga Valley Development.

MAURICE STILES PARKER, M. Am. Soc. C. E.*

DIED JUNE 3, 1921.

Maurice Stiles Parker was born at Groveland, Mass., on December 3, 1851. He received his education at the West Point Military Academy and the University of Berlin, which he entered in 1871, and where he studied civil and mechanical engineering for three years. Before leaving Europe, he visited many of the important engineering works on the Continent.

Returning to the United States, in the spring of 1875, Mr. Parker established an office at La Crosse, Wis., and carried on a private engineering practice until July, 1877. From July, 1877, to July, 1881, he was Chief Clerk of the Mining Department of Dakota Territory, in the office of the United States Surveyor-General, and from July, 1881, to January, 1882, he was United States Deputy Mineral Surveyor, locating flumes in the Black Hills, Dakota Territory, and Consulting Engineer to Samuel McMaster, Superintendent of the Homestake Mine.

From April, 1882, to June, 1883, Mr. Parker served as Division Engineer in Charge of Construction on the Chicago and Northwestern Railway, and from August, 1883, to April, 1884, he was engaged as Locating Engineer for the Chicago, Burlington, and Northern Railway Company.

He held the position of Acting City Engineer of Winona, Minn., in charge of public improvements, from April, 1884, to April, 1885, and from April, 1885, to March, 1886, he was Resident Engineer of the Chicago and Northwestern Railway. He was employed as Locating Engineer of the Montana Extension of the St. Paul, Minneapolis, and Manitoba Railway from April, 1886, to January, 1888, when he opened an office as Consulting Engineer, at Winona, Minn., serving also as Chief Engineer of the Sioux Falls, De Smet, and Northern Railway.

In 1890, he was appointed Chief Engineer of the Great Falls Water Power and Townsite Company and had charge of the construction of the Black Eagle Dam, at Great Falls, Mont., in 1890-91, which was the first power development on the Missouri River. Later, he became engaged in railroad contracting in Montana and afterward in the States of the Southwest. For a number of years before his death, which occurred on June 3, 1921, at Spokane, Wash., Mr. Parker had made his home at St. Maries, Idaho, where he was engaged in water-power development and drainage projects in Montana and Northern Idaho.

He was married in 1875 in Geneva, Switzerland, to Miss Minneowa Burns, who died in 1917. Mr. Parker is survived by his daughter, Mrs. Charles W. Craney, of St. Maries, Idaho.

Mr. Parker was elected a Member of the American Society of Civil Engineers on February 5, 1890.

* Memoir compiled from information on file at the Headquarters of the Society.

CORNELIUS VAN VORST POWERS, M. Am. Soc. C. E.*

DIED JUNE 18, 1922.

Cornelius Van Vorst Powers, the son of William P. and Mary B. (Van Vorst) Powers, was born in Jersey City, N. J., on November 4, 1860. He received the degree of Ph. B. from the School of Mines, Columbia University, New York City, in 1882. After his graduation, he spent some time traveling in Europe and was engaged in work (other than engineering) until February, 1885.

Mr. Powers began his engineering career with the Aqueduct Commission of New York City, then engaged in constructing the so-called New Croton Aqueduct from Croton Dam to New York City. He served with this Commission until June, 1900, rising successively from junior engineering positions to that of Assistant Engineer. He was engaged on the construction of the Aqueduct Tunnel in the upper part of the Borough of Manhattan, New York City, and had charge of the construction of a part of the pipe line, consisting of a battery of 48-in. pipes, eight in number, where it left the terminal gate-house at 135th Street and Convent Avenue, four of the pipes continuing to the reservoir in Central Park. He also had charge of a blow-off from Pressure Shaft No. 24 to the Harlem River. The experience gained in the building of this great tunnel was reflected in his work on the New York City Rapid Transit construction with which he was later connected. On the completion of the pipe line and blow-off, he was assigned to the Croton River Division in Westchester County, where he had charge of the surveys and studies for additional reservoirs and for maps for the condemnation of lands to be acquired, as well as for the necessary relocation and reconstruction of railroads, highways, and highway bridges, and the estimates of cost.

In June, 1900, Mr. Powers was transferred to the Engineering Staff of the Board of Rapid Transit Railroad Commissioners, New York City, at the time when work was begun on the construction of the first subway, and served with that Commission and its successors, the Public Service Commission, First District, State of New York, the Transit Construction Commissioner, and the Transit Commission, on the work of constructing the New York City Rapid Transit Railroads. His first work on rapid transit construction was as Assistant Engineer in charge of the construction of the two-track subway on Broadway, from 135th Street to Dyckman Street, a large part of which was in deep rock tunnel, including two arched tunnel stations with shafts, in which were installed elevators and stairways providing access from the surface. At these stations, the tunnel was excavated wide enough to span the two tracks and the side platforms of the stations.

Mr. Powers was appointed Division Engineer in March, 1903, and, until January 1, 1917, he supervised the construction of the subway and elevated

* Memoir prepared by Robert Ridgway and Charles D. Searle, Members, Am. Soc. C. E.

lines in Upper Manhattan and The Bronx, which were built under contract with the Commissions. He also supervised the surveys for new rapid transit lines, as well as the construction and the certification of the reasonable cost of the third-tracking and extensions of the lines of the Manhattan Elevated Railroad Company in these Boroughs, the latter work being done by the Company under certificates approved by the Public Service Commission. During this period, Mr. Powers also assisted in the preparation of engineering data for the city's defense in the arbitration proceedings growing out of the contractors' claims on the contract for the original subway, as well as in the writing of the contracts and specifications for certain later lines, covering construction in Lower and Upper Manhattan and in The Bronx.

As Division Engineer in Charge of Construction, the value of the work which Mr. Powers supervised approximated $35 000 000, and included deep and difficult tunnel and subway construction, some of which was adjacent to closely built-up sections in the city, as well as the construction of a two-track tunnel under the Harlem River, which was built by novel methods, and another four-track tunnel under this same river, which was constructed in a manner somewhat similar to that used for the tunnel of the Michigan Central Railroad under the Detroit River. The value of third-tracking and extension work of the lines of the Manhattan Elevated Railroad Company approximated $20 000 000.

Mr. Powers' work in connection with the city's defense in the arbitration proceedings, as well as his work in the writing of the contracts and specifications, was so thorough and fair that it has been spoken of in high terms by those who were familiar with it.

From January, 1917, he was the Division Engineer in Charge of the Bureau of Contract Adjustment, which position he held at the time of his death on June 18, 1922. The result of the painstaking, minute, and clear analysis of the contractors' claims which he made personally and directed during the time he was the head of this Bureau, have been and will be of great value to the city.

Mr. Powers was a man of modest disposition, high ideals, and absolute integrity; he was conscientious almost to a fault. He was well informed, with a keen sense of humor, and was a good friend, always ready to help in time of misfortune. He was held in high and affectionate esteem by his associates. It has been deservedly said of him that he was a Christian gentleman. He was fond of outdoor life, an ardent fisherman, and had a great fondness for flowers, which he cultivated with much pleasure. He was an excellent amateur photographer and, in the earlier part of his career, delighted in taking pictures of points of interest on his work and, later, on his fishing trips. An inspection of the reports of the Aqueduct Commission will show many good examples of his work among the illustrations. He worked with his usual zest and interest until the evening of the day before he was stricken.

Mr. Powers was unmarried and is survived by his brother, William Van Vorst Powers.

He was a member of the University Club of New York City, Columbia University Club, Psi Upsilon Fraternity, American Association of Engineers, Municipal Engineers of New York, of which he had been Vice-President and Director, and the Dutch Reformed Collegiate Church of St. Nicholas, New York City, in the activities of which he took great interest.

Mr. Powers was elected a Member of the American Society of Civil Engineers on March 1, 1905.

LINGAN STROTHER RANDOLPH, M. Am. Soc. C. E.*

DIED MARCH 7, 1922.

Lingan Strother Randolph was born on May 13, 1859, at Martinsburg, Va. He was the son of James L. Randolph and Emily Strother Randolph, and was directly descended from William Randolph who settled at Turkey Island, Virginia, soon after the settlement at Jamestown, in 1607. Several members of the family distinguished themselves in the Engineering Profession. His father, the late James L. Randolph, M. Am. Soc. C. E., was Chief Engineer of the Baltimore and Ohio Railroad for many years, and his brother, Beverly Randolph, was Chief Engineer of the Consolidation Coal Company.

Lingan Strother Randolph received his early education in private schools and at the Virginia Military Institute. He also served an apprenticeship at the shops of the Baltimore and Ohio Railroad at Grafton and Mt. Clare, Md. In 1881, he entered the Junior Class at the Stevens Institute of Technology and was graduated with the degree of Mechanical Engineer in 1883. At the Forty-ninth Annual Commencement, on June 21, 1921, he received from the Institute the Honorary Degree of Doctor of Engineering.

From 1883 to 1885, Mr. Randolph was employed by the Erie Railway at Susquehanna, Pa., as Engineer of Tests. He held the position of Superintendent of Motive Power of the Florida Railway and Navigation Company from 1885 to 1887, at which time he left Florida to become Superintendent of Motive Power of the Cumberland and Pennsylvania Railway, which position he retained until 1890. He was employed as Engineer of Tests on the Baltimore and Ohio Railroad from 1890 to 1892, and as Electrical Engineer by the Baltimore Electric Refining Company from 1892 to 1893.

As Professor of Mechanical Engineering and, later, as Dean of the Engineering Department, he was a member of the Faculty of the Virginia Polytechnic Institute from 1893 to 1918. It was during his twenty-five years at the Virginia Polytechnic Institute that Professor Randolph rendered his greatest service to the Engineering Profession. Those trained under him not only received a thorough knowledge of the profession, but were given a practical working knowledge of the problems that would be met in later life. The students were also trained in the highest ethics of the profession.

Of his work at the Virginia Polytechnic Institute, Dean Theodore P. Campbell, writes:

"Professor L. S. Randolph succeeded, as Professor of Engineering, the late James H. Fitts, who was unfortunately killed in a railroad accident on his way to the Chicago Exposition. If my recollection serves me aright, Professor Randolph was suggested to the President of the Institution by the President of the Baltimore and Ohio Railroad.

"He entered on his duties as Professor of Engineering in September, 1893. The Engineering Department was subsequently developed and divided into Mechanical, Electrical, and Civil Engineering—Mining Engineering being introduced many years later. When the above division took place, Professor

* Memoir prepared by H. A Gillis, Esq., Washington, D. C.

Randolph was made head of the Mechanical Engineering Department and had charge of the college shops and of the college repair work. He remained head of the Mechanical Engineering Department until his resignation was tendered in 1918.

"During the latter part of President Barringer's administration, Professor Randolph was made Dean of the whole Engineering Department. This office he filled until his connection with the institution was terminated by his resignation.

"Professor Randolph brought to the institution a wide knowledge of men in the professions and in the business world. He was instrumental in making this institution known to the various engineering and business activities. It was he who brought it into actual connection with the engineering societies. He also possessed, in a large degree, and emphasized with the students, the human element in engineering. The fact that Professor Randolph was highly esteemed and valued by these various men and organizations was brought to my attention as Dean of the institution. As a professor, he was very popular with his classes, and I have often noticed, as have others of his associates, that, when the old men returned here, it was almost invariably Professor Randolph they asked for first, showing the affection and esteem in which they held him.

"Professor Randolph's activities were, however, in no sense, limited to the classroom. He was largely responsible for putting the Y. M. C. A. on a secure footing at the V. P. I. and for maintaining it there. He was generous almost to a fault and extremely liberal in his contributions to the Y. M. C. A.

"He was an Elder in the Presbyterian Church in this town, in which he retained his membership until his death. He was very much interested in all the interests and activities of the church and for a long period of years taught a Bible Class in the Presbyterian Sunday School and also conducted a Normal Class in the Y. M. C. A.

"To him also, more than to any other one man, and almost entirely to him, I may say, was due the fact that what is now a branch of the Norfolk and Western Railroad connecting Blacksburg with the main line of the Norfolk and Western at Christiansburg, was built and operated.

"As a man, he was what may be termed a 'big man'—a man of big heart and big ideas. I was intimately associated with him for 24 or 25 years and can truly say that I never saw anything in Professor Randolph's actions that smacked the least of pettiness. He was a faithful friend, a genial companion, an earnest Christian, a useful citizen, and his influence in any community in which he lived could not but be helpful and elevating."

Professor Randolph found time from his duties at the Virginia Polytechnic Institute to perform a great deal of important consulting work. He also wrote a number of papers and discussions for the Society and for the other engineering societies of which he was a member, among which were the American Society of Mechanical Engineers, the International Association for Testing Materials, the American Institute of Electrical Engineers, and the Tau Beta Pi Fraternity.

During the World War, he served in the Research Section of the U. S. Shipping Board, Emergency Fleet Corporation, and the latter part of his life was spent in looking after his special interests, among which was the Brush Mountain Coal Company of which he was President.

He was married on October 15, 1890, at Cumberland, Md., to Fanny Robbins, who, with three sons and one daughter, survives him.

Professor Randolph was elected a Member of the American Society of Civil Engineers on January 2, 1890.

KARL DE WITT SCHWENDENER, M. Am. Soc. C. E.*

DIED JANUARY 22, 1922.

Karl De Witt Schwendener was born at Cleveland, Ohio, on April 25, 1885. He was educated at Harvard University, from which he was graduated in 1908 with the degree of Bachelor of Science.

During the summer vacation of 1907 he was employed as Levelman on the Buffalo, Rochester and Pittsburg Railway. After his graduation from Harvard, Mr. Schwendener went to Los Angeles, Calif., and from December, 1908, to May, 1910, was engaged as Draftsman, and, later, as Chief Engineer of the California Building and Land Syndicate, in charge of engineering work relating to building construction, surveys, and sub-division of properties.

From June to December, 1910, he served as Designing Engineer for the Los Angeles Railway Corporation, for which Corporation he designed, among other structures, reinforced concrete carshops and two carhouses.

In January, 1911, Mr. Schwendener was appointed Assistant Inspector of Buildings of the City of Los Angeles and had charge of the inspection of steel frame and brick construction.

In May, 1912, he was appointed Chief Engineer of the Department of Buildings of the City of Los Angeles, in charge of all the inspection and engineering work of the Department. In addition to ten fire-engine houses and various other city buildings which were constructed under his supervision, he assisted in the preparation of revised building ordinances and State laws. He also prepared a treatise on the design of wood beams and columns, containing a series of tables, which has become a textbook in the Department, and among the architects and builders of Los Angeles.

During the World War, Mr. Schwendener was commissioned a Captain in the Engineer Corps, and served overseas where he was in charge of construction work. He was on the firing line in the Argonne and, for three months, with the Army of Occupation in Germany.

He died of pneumonia on January 22, 1922, after a brief illness, and is survived by his wife and three children.

Mr. Schwendener was an active member of the Los Angeles Chamber of Commerce, the City Club, and American Legion Post 110.

The high esteem in which he was held by his fellow engineers of the Los Angeles Section of the Society, in which he took an active part, is shown in the following resolution which was adopted at the meeting of the Section on April 12, 1922:

"*Whereas*, it has pleased the Almighty to take from us our beloved fellow-member and associate, Karl D. Schwendener; and

"*Whereas*, through his going away we have sustained the loss of an able brother engineer, adviser, and a friend whose place will never be filled. Possessing those strong sterling qualities of manhood and professional integrity that won for him the high place in the counsels of this Society and those of

* Memoir prepared by John E. Rockhold, Assoc. M. Am. Soc. C. E., and F. G. Dessery, M. Am. Soc. C. E., and from information on file at the Headquarters of the Society.

us who knew him best, we feel an irreparable loss of a loyal comrade and associate we had learned to love, respect, and honor;

"*Therefore, Be It Resolved,* that these resolutions be entered upon the permanent records of the Los Angeles Section, American Society of Civil Engineers, and a copy be sent to his bereaved family, with our heartfelt sympathy in their affliction."

Mr. Schwendener was elected an Associate Member of the American Society of Civil Engineers on April 1, 1914, and a Member on November 25, 1919.

HARRY KENT SELTZER, M. Am. Soc. C. E.*

DIED DECEMBER 30, 1921.

Harry Kent Seltzer, the son of William K. Seltzer and Emma (Keller) Seltzer, was born at Ephrata, Pa., on January 22, 1875. Nearly all his ancestors on both sides were Pennsylvania farmer folk.

Immediately after his graduation from Lehigh University, where he received the degree of Civil Engineer in 1895, he entered the service of J. A. L. Waddell, M. Am. Soc. C. E., and went to Sioux City, Iowa, as Assistant to Lee Treadwell, M. Am. Soc. C. E., Resident Engineer on the construction of a bridge over the Missouri River. During the early part of 1896 he was Inspector of Superstructure on a bridge over the Missouri River at Jefferson City, Mo. In May, 1896, he entered the employ of the Kansas City Southern Railroad, as Instrumentman and Draftsman on preliminary and location work in Southwestern Louisiana. During a portion of this time he was Inspector on the foundations for the Calcasieu River Bridge at Lake Charles, La., on this line.

From May to November, 1898, he was Transitman on Maintenance of Way work, on the Philadelphia Division of the Pennsylvania Railroad, at Philadelphia, Pa. During the college year 1898-99 he was Instructor in Civil Engineering at the University of Texas, after which he spent six months as Resident Engineer for Waddell and Hedrick on the construction of bridges along the line of the Vera Cruz and Pacific Railway, in the State of Vera Cruz, Mexico. This was during the beginning of construction of that road, and conditions were very primitive. While there, Mr. Seltzer almost succumbed to a very severe attack of yellow fever.

After a year spent in Kansas City as Designer and Contracting Engineer for the Midland Bridge Company, he returned to the employment of Waddell and Hedrick, in April, 1901, with whom he was associated until May, 1906. During this period he was stationed first at Alexandria, La., as Resident Engineer on a bridge over the Red River. In May, 1902, he went to New Westminster, B. C., Canada, in charge of the construction of the Fraser River Bridge. This structure was notable for the main pier foundations sunk by open dredging methods in 60 to 80 ft. of water to a depth of 138 ft. below the water surface, and in a current, at times, extremely rapid.

While at Westminster, Mr. Seltzer was also in charge of work for the Great Northern Railway at Vancouver, B. C., Canada, all of which reflected great credit on his resourcefulness and executive ability. During the year following May, 1905, he was in charge of the reconstruction of ten bridges on the International and Great Northern Railroad, in Texas, where the work was done under traffic.

In May, 1906, Mr. Seltzer was employed by the Missouri Valley Bridge and Iron Company, Contractors for the foundations of the Atchafalaya Bridge,

* Memoir prepared by L. S. Stewart, Affiliate, Am. Soc. C. E., E. E. Howard, and M. B. Case, Members, Am. Soc. C. E., and S. E. Blum, Esq., Detroit, Mich.

at Morgan City, La., for the Southern Pacific Railway Company. Here he was in charge of sinking pneumatic caissons to a depth of 105 ft. in 70 ft. of water, and with a very soft bottom which developed little supporting friction.

At the conclusion of this work, Mr. Seltzer became a Member of the Union Bridge and Construction Company, and from 1907 to 1912, as Constructing Engineer, he contributed in no small way to the successful execution of many large contracts. At Portland, Ore., he directed the construction of the foundations for the Oregon-Washington Railroad and Navigation Company bridge over the Willamette River, where the main piers were sunk by open dredging methods in 85 ft. of water to a depth of 133 ft. below the water surface. The total cost of the substructure on this contract exceeded $500 000. During 1911 and 1912 he was stationed in Kansas City, but made frequent trips to Portland where he directed the work on the foundations for the Broadway Bridge over the Willamette River. This substructure consisted of four main piers, sunk by the pneumatic process, and other open foundations at a total cost of $750 000. The contract for the foundations of the bridge over the Yellowstone River, at Mondak, Mont., for the Great Northern Railway Company, was also completed during this time.

Mr. Seltzer's capacity to handle men and materials and to draw about him and hold those whose ability, loyalty, and earnestness were beyond question was now recognized by his promotion to the position of Vice-President and Chief Engineer. In the period from 1913 to 1918 the firm constructed the foundations for three of the largest bridges built in this country during that time—the Harahan Bridge, over the Mississippi River at Memphis, Tenn., the crossing of the Ohio River at Metropolis, Ill., and the Burlington Railroad Bridge over the Missouri River at Kansas City. The complete satisfaction frequently expressed by the officials of the railroads for whom these works were executed reflected in no small degree the untiring zeal and resourceful ability which Mr. Seltzer constantly applied to his work. The reputation for rugged honesty and efficient production which he and the organization he built around him enjoyed has often been a subject of comment, and was shown by the frequency with which he was asked to carry out force-account work for various railroads, with little or no competition.

During the war period, 1917-19, he was stationed at Morgan City, La., as Manager of the Emergency Fleet Corporation's shipyard, and built nine ships, every one of which was put into commission. This yard was a source of great satisfaction to the Shipping Board officials at a time when many were not operating so smoothly.

In November, 1920, Mr. Seltzer severed his connection with the Union Bridge and Construction Company, and represented the Foundation Company of New York at Kansas City.

He died suddenly on December 30, 1921, at Kansas City, Mo., only a few days after his return from a business trip in the East.

One of his friends writes:

"I was Mr. Seltzer's assistant in Alexandria in 1901 and 1902, and we lived together there, and since that time we have been more or less in contact throughout all his work. Throughout his entire career he always commanded the confidence and respect of every one with whom he came into contact. He was so universally esteemed that I cannot recall ever hearing any one say anything derogatory about him, and I do not think I have another acquaintance concerning whom I have not heard some criticism."

Mr. Seltzer was married in November, 1906, to Miss May E. Waters, who, with two children, survives him.

He was a member of The University Club of Kansas City, the Delta Tau Delta Fraternity, and the American Association of Engineers.

Mr. Seltzer was elected a Junior of the American Society of Civil Engineers on February 4, 1896; an Associate Member on January 2, 1901; and a Member on April 3, 1906.

EDWARD BALLINGER TAYLOR, M. Am. Soc. C. E.*

DIED NOVEMBER 8, 1922.

Edward Ballinger Taylor was born at Riverton, N. J., on February 6, 1850. He was a son of John Gardiner Taylor and Rebecca Haines (Ballinger) Taylor. His parents were members of the Society of Friends, and he was trained in the tenets of that faith. The ideals of integrity, devotion to duty, and kindliness, inculcated by that training were reflected in his character through his whole life.

Mr. Taylor was prepared for college at the Westtown, Pa., Boarding School, and in 1866 entered the Sophomore Class at Haverford College, from which he was graduated in 1869, receiving the degree of Bachelor of Arts. He then entered the Polytechnic College of the State of Pennsylvania where, in 1870, he was given the degree of Bachelor of Civil Engineering. From the same institution, in 1873, he received the degree of Master of Civil Engineering.

On July 25, 1870, Mr. Taylor entered the service of the Pennsylvania Railroad, as a Clerk in the office of the Superintendent of the Middle Division, at Harrisburg, Pa. His efficient service won rapid advancement, and in September, 1871, he was promoted to the position of Supervisor, and six months later, on March 1, 1872, was appointed Assistant Engineer of the Middle Division. He continued in that office until January 1, 1875, when he was transferred to a similar position on the Pittsburgh Division.

On July 24, 1876, just six years after entering the service of the Pennsylvania Railroad, he was made Superintendent of the Lewistown Division, with headquarters at Sunbury, Pa., where he remained until January 1, 1879, when he was transferred to the Superintendency of the West Penn Division. On September 1, 1879, he was transferred to the Lines West of Pittsburgh, and made Superintendent of the Pittsburgh, Cincinnati and St. Louis Railway. He was promoted on April 1, 1888, to the position of General Superintendent of the Northwest System, and on March 1, 1890, to that of General Superintendent of Transportation of the entire system of the Lines West of Pittsburgh.

On December 27, 1901, Mr. Taylor was elected a Director and Fourth Vice-President of the Pennsylvania Company and of the Pittsburgh, Cincinnati, Chicago and St. Louis Railway Company. On January 9, 1907, he became Third Vice-President, and on February 1, 1914, Second Vice-President, in charge of the Treasury and Accounting Departments. He also served as President, Vice-President, or Director of fifty-two subsidiary or affiliated companies of the Pennsylvania System. On March 1, 1920, having attained the age of seventy years, he was, in accordance with the provisions of the Pension Department of the Pennsylvania System, retired from active service.

* Memoir prepared by W. L. R. Haines, M. Am. Soc. C. E.

In addition to his duties as an officer of the Pennsylvania Railroad, Mr. Taylor was for more than twenty-five years a Director of the First National Bank of Sewickley, Pa., and was for some time a Director of the Second National Bank of Pittsburgh, Pa., and, after its merger, of the First-Second National Bank of Pittsburgh, Pa.

At the time of his death, Mr. Taylor had been for many years a resident of Sewickley, Pa., and had always taken a keen interest in its needs and activities. Since 1891, he had served as a member, and since 1895, as President, of its Board of Water Commissioners, and it was due largely to his efforts and guidance that the Sewickley water supply was brought to its present status, as one of the best and most economical in the State of Pennsylvania. He was one of the original members of the Board of Trustees of the Sewickley Valley Hospital and of the Sewickley Young Men's Christian Association, and took an active part in the work of both institutions. He had served as President of the Corporators of the Sewickley Cemetery and of the Edgeworth Club. He was a member of the Allegheny Country Club, the Pittsburgh Club, and the Bankers Club of New York. He was also a member of the American Academy of Political and Social Science, and of the Pittsburgh Academy of Science and Art.

He was a Charter Member of the Engineers' Society of Western Pennsylvania, and was President of that Society in 1886, and a Director in 1887 and 1888.

Although Mr. Taylor never forsook his allegiance to the Society of Friends, he was for many years a communicant of St. Stephen's Protestant Episcopal Church in Sewickley. He served as a Vestryman and as a member of the Building Committee of this church, and was largely instrumental in securing the replacement of the old frame building with the present handsome stone edifice. In his later years, he attended the Sewickley Presbyterian Church, of which he was twice elected a Trustee.

Mr. Taylor was married in 1872 to Miss Marianna Satterthwaite, of Oxford Valley, Pa., by whom he is survived. He also leaves three daughters, Mrs. Charles A. Woods, Miss Bertha A. Taylor and Miss Rebecca W. Taylor, all of Sewickley, and one son, Edward B. Taylor, Jr., Assoc. M. Am. Soc. C. E., Superintendent of the Marietta Division of the Pennsylvania Railroad, with headquarters at Cambridge, Ohio.

It seems not inappropriate to close this memoir with a quotation from the editorial columns of a local paper—a fitting tribute to the regard in which Mr. Taylor was held by those among whom he lived and worked:

"There are but few men in the Sewickley Valley whose death would be so great a public loss as is that of Edward B. Taylor. Known and liked in a personal way by every one, he was regarded by all with a most unusual respect because of his long and valuable service to Sewickley and to the Valley. The same skill, knowledge, broad vision, and safe judgment which brought him pre-eminence in the management of a great railroad system's affairs, had for many years been placed freely at the service of his home community and its needs, and have been instrumental in contributing much to its welfare and

will contribute much more in the future as improvement plans not yet brought to completion are worked out by other hands. Few men have given so freely, so effectively and so unobtrusively of their time and wisdom in a constructive way to the guiding and betterment of public undertakings in their home communities, and Mr. Taylor's death is a loss indeed to this Valley."

Mr. Taylor was elected a Member of the American Society of Civil Engineers on September 3, 1884.

SAMUEL CLARENCE THOMPSON, M. Am. Soc. C. E.*

DIED FEBRUARY 28, 1920.

Samuel Clarence Thompson, the son of Samuel Newman Thompson and Mary Ann (Washburn) Thompson, was born in Roxbury, Mass., on April 4, 1851. The family moved from Roxbury to Southboro, Mass., while Mr. Thompson was in his early boyhood. He won a scholarship in the Massachusetts Agricultural College and was graduated therefrom in 1872.

In 1875, Mr. Thompson was engaged as an Assistant Engineer on the new Water Supply Commission of the City of Boston, and in 1879 was appointed as Acting Chief Engineer on the construction of the South Florida Railroad. Later, in 1881, he was made Assistant City Engineer of Lowell, Mass., and, in this capacity, had charge of the laying of the large supply main from the high-service reservoir in Centralville, under the Merrimac River, to Belvedere. Subsequently, he was in charge of rebuilding the bridge across the Merrimac River.

Mr. Thompson came to New York City in 1883 and entered the service of the municipality in the Department of Parks, in the Borough of the Bronx, which, at that time, was known as the 23d and 24th Wards. He continued his connection with the City of New York in the Department of Highways, through its changes of name and jurisdiction, until his death on February 28, 1920.

Mr. Thompson was in the service of the city for 37 years and, until his last illness, lost only 3 days from his work during that time. He was in charge of the Bureau of Highways in the Office of the Borough President of the Bronx for many years, and there are numerous monuments in the Borough to his ability and energy as an Engineer. He was also in charge of the regulating, grading, and paving of most of the streets in the West Bronx, of the construction of the Grand Boulevard and Concourse, and of all its connecting transverse roads.

On May 5, 1875, he was married to Miss Alice Louise Fairchild of Natick, Mass., who, with a daughter, Mrs. Elsie Thompson Shepard, survives him.

Mr. Thompson was a Charter Member and a Past-President of the Municipal Engineers of the City of New York. He was also a member of the National Geographic Society and the League of American Sportsmen, a Knight of Honor, a member of the Royal Arcanum and of the Loyal Association of the Royal Arcanum, of Roome Lodge No. 746, F. and A. M., Chancellor of the Supreme Court of the Phi Sigma Kappa, and a member of the Phi Kappa Phi.

His loss was deeply felt by the Engineering Corps in the Office of the President of the Borough of the Bronx and by all his associates and friends.

Mr. Thompson was elected a Member of the American Society of Civil Engineers on February 6, 1889, and served as a Director from 1909 to 1911.

* Memoir prepared by Josiah H. Fitch, M. Am. Soc. C. E.

SAMUEL EVERETT TINKHAM, M. Am. Soc. C. E.*

DIED APRIL 21, 1921.

Samuel Everett Tinkham, the son of Samuel Miller and Celia (Ellis) Tinkham, was born at Taunton, Mass., on March 31, 1852. He was a descendant of a long line of New England ancestors, some of whom came to New England in the *Mayflower*. His father was Treasurer of the Taunton Foundry and Machine Company, and afterward established a music store in Taunton, which he conducted for many years. Mr. Tinkham's mother died when he was only three years of age. His father afterward married Miss Margaret E. Bright, who, having been a teacher, was well qualified to direct the education of her four stepsons.

It was through her efforts, that Mr. Tinkham was able to qualify for the Massachusetts Institute of Technology and to become a member of the Class of 1873. During the summer of 1872, he was engaged in active field work as Rodman in the Engineering Department of the New York State Canals. During the summer following his graduation, he was appointed an Assistant in the Corps of Engineers, United States Army, and for more than a year was employed on harbor improvements at Edgartown, Mass.

In October, 1874, Mr. Tinkham was appointed as Transitman in the office of the City Engineer of Boston, Mass. In addition to his regular work, he made experiments with the blue-print process, then in its infancy, and thus helped to develop and to make the process of reproduction by blue-printing what it is to-day.

During the first eight years of his service with the City of Boston, from 1874 to 1882, Mr. Tinkham was engaged principally on bridge work, under the late John E. Cheney, M. Am. Soc. C. E., who was then City Bridge Engineer. At this time, several city bridges were built or reconstructed, among which were the Dover Street, Meridian Street, Broadway, and Chelsea Bridges. Part of Mr. Tinkham's time was also given to work in connection with the Boston Main Drainage System, especially with the design and construction of the Calf Pasture Pumping Station.

In the fall of 1882, Mr. Tinkham left the employ of the City Engineering Department with the idea that he could find a more promising future elsewhere. He became Assistant Engineer on the New York and New England Railroad, in charge of the design and erection of bridges for the double-tracking of that line. His service with this road was terminated in April, 1884, when he returned to Boston and again entered the office of the City Engineer, where he remained for thirty-seven years, until his death.

On his return to municipal work, Mr. Tinkham was given the title of Assistant Engineer and Principal Draftsman of the Engineering Department. He became Chief Assistant to the Bridge Engineer, and his work was largely in charge of bridge designing. Among the bridges constructed at this time were the Allston, Beacon Street, Boylston Street, Brookline Avenue, and

* Memoir prepared from information on file at the Headquarters of the Society.

Cottage Farm Bridges, over the Boston and Albany Railroad; the Byron Street Bridge, in East Boston; the Dover Street and L Street Bridges, in South Boston; the draw-span of the Neponset Bridge; and the Harvard Bridge across the Charles River, which was built jointly by the Cities of Boston and Cambridge, under the direction of the Boston Engineering Department. In addition to bridge work, Mr. Tinkham was also engaged on the Boston Improved Sewerage System, the Mystic Pumping Station, and on work in connection with the elimination of grade crossings.

During the late Nineties, several new bridges were built, the most important of which were the Charlestown and Cambridge Bridges. The South Terminal Station, which was to unite the railroads on the south side of the city, was also projected about this time and involved much planning, including the rearrangement of the streets and extensive constructions along the water-front of Fort Point Channel. Mr. Tinkham was directly in charge of the municipal part of this work.

He was engaged, during the last twenty-three years of his service with the City Engineering Department, principally on construction engineering, which included bridges, sea walls and retaining walls, tide-water foundations, harbor shore protection, ferry improvements, wharves, and other water-front structures.

Mr. Tinkham was also connected with other well-known projects, among which were the Northern Avenue and Sleeper Street improvement, from Atlantic Avenue, across Fort Point Channel and the South Boston Flats, involving the construction of one of the most important of the City's tide-water bridges; grade-crossing elimination, including the building of a number of new bridges over railroads; the rebuilding of the Chelsea Bridge north over the Mystic River, including the heaviest swing draw-span in New England; the building of the new swing draw-span of the Broadway Bridge, over Fort Point Channel; the rebuilding of the Meridian Street and Chelsea Street Bridges, between East Boston and Chelsea, each with a swing draw-span; the reconstruction of the Malden Bridge between Charlestown and Everett; and the rebuilding of the Granite Avenue Bridge over the Neponset River.

In 1911, the City Engineering Department, the Street Department, and the Water Department, of Boston, were united to form the present Public Works Department, and Mr. Tinkham was made Construction Engineer of the Bridge and Ferry Division.

In addition to his work for the city, he was frequently called on as Consulting Engineer. In 1897, he was appointed as one of the members of the first Board of Examiners for Civil Engineers in the classified service, a position which he held for more than fifteen years, until January, 1913.

Mr. Tinkham was primarily a Bridge Engineer, but he was also an authority on foundation problems, particularly on constructions in tide-water. He was absolutely honest, of a cheerful disposition, and was an agreeable companion. He died of apoplexy at his home in Roxbury, Mass., on April 21, 1921.

Mr. Tinkham was married on October 20, 1879, to Miss Louisa W. Danforth, of Roxbury, who, with three children, Charles S. Tinkham, Resident Engineer of the Massachusetts Highway Commission at Greenfield, Mass., Frank B. Tinkham, of Boston, and Mrs. Leo Twombly, of East Orange, N. J., survives him.

He was Secretary of the Boston Society of Civil Engineers for thirty-six years, and his efforts in behalf of the interests of that Society were indefatigable and highly appreciated by every member. Its successful history and rapid development were due largely to his untiring and self-sacrificing interest. He was a member of the New England Water Works Association and the Massachusetts Highway Association.

Mr. Tinkham was elected a Member of the American Society of Civil Engineers on March 2, 1892.

WILLIAM DAVID UHLER, M. Am. Soc. C. E.*

DIED OCTOBER 27, 1922.

William David Uhler, the son of the late Andrew Jackson Uhler and Susan E. Seiple Uhler, was born at Nazareth, Pa., on November 8, 1872. He received his preparatory education in the public schools of Easton, Pa., having been graduated from the High School in June, 1889. In June, 1910, the Maryland Agricultural College, now the Maryland State University, conferred on him the honorary degree of Civil Engineer, in recognition of his work with the Maryland State Roads Commission.

During summer vacations, and prior to his graduation from High School, Mr. Uhler was employed on Corps work with the Lehigh Valley Railroad Company, and, on the completion of his High School course, he entered the service of this Company, under Mr. I. W. Troxel, as Chainman, Rodman, and Instrumentman. In 1892, he became Assistant Engineer of the Baltimore and Lehigh Railroad Company, under Mr. Troxel, at Baltimore, Md., but returned to the Lehigh Valley Railroad Company in 1893 as Transitman, remaining with that Company until 1895. During these periods, he was employed on preliminary and location surveys and on construction work.

In 1895, Mr. Uhler returned to Baltimore as Assistant Chief Engineer of the Delaware, Maryland and Virginia Railroad Company (the Queen Anne Railroad), in which capacity he was employed on the preliminary and location surveys and the construction of this line. He retained this position until the road was ready for operation in 1897, and from 1897 to 1904, he served the Company as Assistant General Manager. His ability as an Engineer and an organizer was early recognized in his association with this Company, and his superior officer states that, in the later years of his services with this line, "he had full charge of all the Company's affairs."

In 1904, he was made County Road Engineer of Caroline County, Maryland, in charge of the construction and maintenance of highways and bridges. During his incumbency of this office, which was in the earlier days of highway development, he originated methods of design and maintenance, and under his tutelage and able executive direction an organization was developed, which gradually placed this work under business management and administration.

In 1908, under W. W. Crosby, M. Am. Soc. C. E., then Chief Engineer of the Maryland State Roads Commission, Mr. Uhler accepted the position of Assistant Engineer of the Maryland State Roads Commission. He remained with the Commission from 1908 to 1912, having had charge of survey work, the preparation of plans, and the details of the maintenance of the highways of that State. During this time, he organized the maintenance forces throughout the State, put into effect the patrol system of maintenance, and most effectually performed all these duties, in addition to having charge of all

* Memoir prepared by H. E. Hilts, Assoc. M. Am. Soc. C. E.

surveys and all drafting, and similar work, under the Chief Engineer. During this period, in the words of his Chief, "his brilliant mind, laudable ambition, unflagging energy, steadfast loyalty, and untiring application to duty, enabled him to succeed where many others would have failed, and his pleasing personality endeared him to all. Unquestionably, the present high standing, among their users, of the State roads of Maryland to-day, is due to the working out of the details of this maintenance by William D. Uhler."

In 1912, Mr. Uhler was recommended by Mr. Crosby for the position of Principal Assistant Engineer of the Bureau of Highways of Philadelphia, Pa., under the Blankenburg Administration. Mr. Uhler was the successful candidate by Civil Service examination for this post. From 1912 to 1915, he was engaged in the design, construction, and maintenance of city streets and of special park work. The Director of Public Works, Mr. Morris L. Cooke, states that during this time:

"There was not anything that went on in the Bureau to which he did not make a real contribution. It was due, in a large measure to his capacity for detail, his indefatigableness, and his painstaking regard for the matters that must accompany public work, that we were able to accomplish as much as we did in transferring the Department from the point where we employed one engineer out of a total of upward of a thousand employees, to where all of its work practically was carried on under engineering methods and for the most part by engineers. He had the capacity for telling citizens and others what was going to happen, to do it promptly, and in a manner which was inspiring in the confidence of the public."

In 1915, Governor Martin G. Brumbaugh appointed Mr. Uhler as Chief Engineer of the Pennsylvania State Highway Department, and from the date of his appointment to January 24, 1918, when he was granted a leave of absence, his experience, energy, resourcefulness, and executive capacity were engaged in standardizing the maintenance and construction of the highways of the Commonwealth, and perfecting the nucleus of an organization which was to carry on during the World War.

On January 24, 1918, Mr. Uhler was granted a leave of absence by Governor Brumbaugh, and was commissioned a Major in the Quartermaster Corps, Motors Division, U. S. Military Service. On May 8, 1918, he was promoted to the rank of Lieutenant Colonel. He was discharged from the service on November 23, 1918, and on March 29, 1920, he accepted a commission as Colonel in the Quartermaster Section, Reserve Corps. George W. Goethals, Maj.-Gen., U. S. A. (*Retired*), M. Am. Soc. C. E., states of Colonel Uhler's activities during this period as follows:

"In January, 1918, when I took charge of all transportation for the War Department, it was decided to move supplies by motor trucks, where this could be done to advantage. This brought to the fore the necessity for the repair of old, and the construction of new, highways, and in connection with this work, I drafted the services of Colonel Uhler. This work was then expanded to determine whether new construction desired by States or municipalities would arise in war activities, since highway construction was practically stopped during the war period. All questions of this kind, as they concerned

the War Department, in the deliberations of the United States Highway Council, were passed upon by Colonel Uhler, who was thorough in his investigations and development of the facts and whose conclusions showed his ability and foresight. The soundness of his judgment in my earlier association with him soon led me to accept his recommendations without question. He was thoroughly reliable and efficient, performing a difficult task faithfully, conscientiously, and zealously."

On being honorably discharged from the service, Colonel Uhler returned to his work as Chief Engineer of the Pennsylvania State Highway Department, and was re-appointed Chief Engineer by Governor William C. Sproul on April 7, 1920. In association with the late Lewis S. Sadler, State Highway Commissioner, and the Assistant Commissioner, George H. Biles, M. Am. Soc. C. E., Colonel Uhler created and enlarged the State Highway Department and carried on one of the largest post-war programs of State highway construction. Mr. Biles' tribute at the time of Colonel Uhler's death best exemplifies the thoughts of Colonel Uhler's engineering organization:

"The death of Colonel William D. Uhler is an irreparable loss to the State of Pennsylvania. From my many years of association with him I am convinced that he had no peer among highway engineers in this country. In the vast road building program inaugurated by Governor Sproul, he performed such conspicuous service in engineering skill that Pennsylvania to-day leads the nation in the construction of modern highways. His life was dedicated to his ideals in road building to the exclusion of all other things. This intense application soon manifested itself in a weakened physical condition, which he could not overcome, and finally caused his untimely end. I am grieved beyond expression by the loss of such an able and conscientious colleague, and the passing of a sterling character and true friend."

His peculiar fitness for his work may be readily summarized in the words of Thomas H. MacDonald, Chief of the Bureau of Public Roads, United States Department of Agriculture:

"Colonel Uhler was an outstanding figure in the field of Highway Engineering and Administration. His influence was national. He was a leader in the forming of policies and legislation, State and National, to provide adequate modern highways. The accomplishment in engineering of the State Highway Department is a tribute to his leadership. As an organization, the Bureau of Public Roads has respected and appreciated his courage, ability, and integrity of conduct and purpose in his administration of a great public trust. As individuals, we feel most keenly the loss of a real friend."

Colonel Uhler was a member of the American Society of Municipal Improvements, the American Concrete Institute, and the American Society for Testing Materials. He was also a member and a Director of the American Road Builders' Association, and a member of the International Association of Road Congresses and the Engineers' Club of Philadelphia. He was a Past-President and a Director of the American Association of State Highway Officials, having served first as Chairman of its Executive Committee in 1917. He was elected President in 1918, and was re-elected a member of the Executive Committee and served as such until his death. He was also Chairman of the Committee on Standards, and a member of the Committee on Motor Truck Regulation.

Colonel Uhler was married on December 31, 1896, to Marguerite Stevens, daughter of the late Mr. and Mrs. B. S. Stevens, of Denton, Md., who survives him, together with his mother and three sisters.

He was a member of Temple Lodge, F. and A. M., of Denton, Md., Knight Templar Commandery of Easton, Md., Harrisburg Consistory, A. A. S. R., and Boumi Temple, A. A. O. N. M. S., of Baltimore, Md.

Colonel Uhler was elected an Associate Member of the American Society of Civil Engineers on May 6, 1908, and a Member on March 1, 1910.

LUTHER WAGONER, M. Am. Soc. C. E.*

DIED JULY 1, 1922.

Luther Wagoner was a remarkable man. He represented the self-made engineer who had reached the highest standard in theoretical work by reason of intense application and studious habits. Those of his friends who knew him best, admired the facility with which he could recall events and reproduce from memory the details of the technical problems and papers which he had had occasion to study.

Luther Wagoner was born at Connersville, Ind., on May 23, 1847. When he was only 19 years of age, he was appointed Deputy County Surveyor of Johnson County, Missouri. He was engaged on railroad work in 1868, and passed quickly from the position of Rodman to that of Leveler and Engineer in charge of location on various railroad lines in Missouri and Kansas. He also served as Engineer in the Land Department of the Texas Pacific Railroad until 1874, when, with a view to a partnership in a prominent law firm, he began reading law. He did not persist in his law studies, however, preferring to accept an opportunity to go to Brazil on bridge construction work. A year later, Mr. Wagoner was employed as Principal Assistant with the Brazilian Carta Itineraria on geodetic work, and continued thereafter, for a time, in the private practice of engineering in Brazil. In 1878, he went abroad and traveled quite extensively in Europe.

Returning to Kansas in 1879, Mr. Wagoner found employment in the construction of various municipal water-works. In 1880, he yielded to the lure of California and devoted himself thereafter, principally to mining. In 1881, he was engaged as Mining Engineer for the New Almaden Quicksilver Mine (California) and designed and built, for this mine, a mechanical furnace draft. The next year (1882), he designed and built at Murphys, Calif., air classifiers which, at that time, were novelties. For a year, he served as Manager of a mine at Alamos, Mexico, but the call of California was strong, and together with the late James D. Schuyler, M. Am. Soc. C. E., he entered the contracting field and built the rock-fill section, No. 5, of the sea-wall at San Francisco, Calif.

From 1885 to 1890, Mr. Wagoner was active in the practice of general mining engineering, with headquarters at San Francisco. In 1890, he accepted the position of Engineer for the Modesto Irrigation District, and it fell to him to submit a competitive design for the La Grange Dam on the Tuolumne River. This dam, which raises the water level of the river more than 100 ft., serves both the Modesto and the Turlock Irrigation Districts, and the engineers for these Districts were selected to prepare the competitive plans for the dam. The plans were submitted to the late Col. G. H. Mendell, Corps of Engineers, U. S. A., M. Am. Soc. C. E., who recommended the one prepared

* Memoir prepared by Messrs. C. E. Grunsky, William Hammond Hall, and Otto von Geldern, Members, Am. Soc. C. E.

by Mr. Wagoner, with some modifications. Due to some misunderstanding with the management of the District, however, Mr. Wagoner resigned, and the construction work following these modified plans was in charge of other engineers.

Among other work done by Mr. Wagoner in the next few years, the building of a concentrator and power plant for the Washington Reduction Works, the making of plans for an irrigation works on the Lower Colorado River, and the design and construction of works for the Channel Bend Mining Company, in Eldorado County, California, may be noted. In 1900, the City Engineer of San Francisco placed Mr. Wagoner in charge of the field studies and surveys of the available sources of water from the Sierra Nevada Mountains. The outcome of the studies made at that time led to the selection of the Tuolumne River (the Hetch-Hetchy Project) as the most available mountain source of water for San Francisco. From 1902 to 1915, Mr. Wagoner was active on reclamation work—levee building and drainage—and irrigation work in California, New Mexico, and Arizona.

It was during this period that he, with Col. W. H. Heuer, U. S. A. (*Retired*), M. Am. Soc. C. E., was called on by the Federated Harbor Improvement Association of San Francisco to report on the commerce and docks of the harbor and to submit a plan for harbor development. For this study, he went abroad again, and returned particularly impressed with the local lack of port facilities for handling freight, as compared with those at the leading ports of Europe. In 1915, on his selection for the purpose by the Department of State, at Washington, D. C., and the Judge Advocate of the Army, came the call from President Menocal of Cuba to take charge at Havana, Cuba, and adjust the difficulties and controversies which had resulted there in connection with the construction of roads and sewers. Not only did Mr. Wagoner successfully accomplish this mission, but before he returned to California, to recuperate from ills contracted in Cuba (1921), he had the satisfaction of directing the new work that cost between $1 000 000 and $2 000 000.

Mr. Wagoner's attainments in the higher mathematics were particularly noteworthy. He delighted in taking recourse to the calculus in the solution of intricate problems. This appears, for example, in the paper on "Stresses in Curved Masonry Dams", which he contributed, with Mr. Hubert Vischer, to the Technical Society of the Pacific Coast in 1899.

That he had attained a high standing in his profession and commanded the confidence of his fellowmen is sufficiently shown by his selection for the difficult work in Cuba; but as early as 1882 this had already been made evident by his selection by the Governor of California to act as an expert in a controversy relative to the quantity of material placed by the contractors in a section of the San Francisco sea-wall which was being built on a yielding mud foundation.

Mr. Wagoner was a Past-President of the Pacific Association of Consulting Engineers, and of the San Francisco Section of the Society. He was the first President of the Association of Members of National American Engineering Societies in Cuba, and served for many years as a Trustee of the

Mechanics Mercantile Library of San Francisco. He was also a Member of the American Institute of Mining and Metallurgical Engineers and of other organizations.

In his passing, the Engineering Profession has lost a valuable member. His friends miss a delightful and loved companion, and the country has lost a good citizen. He is survived by his widow.

Mr. Wagoner was elected a Member of the American Society of Civil Engineers on May 2, 1906.

FRANK SHERMAN WASHBURN, M. Am. Soc. C. E.*

DIED OCTOBER 9, 1922.

Frank Sherman Washburn, the son of Elmer Washburn, a former President of the National Livestock Bank, of Chicago, Ill., was born on December 8, 1860, at Centralia, Ill.

In September, 1878, he entered Cornell University where be began his engineering studies. From July, 1879, to September, 1880, he was engaged on surveys for the Chicago and Northwestern Railway Company. He then resumed his studies at Cornell, from which he was graduated in June, 1883, having spent the summer months of each year on railroad construction and permanent way work.

In July, 1883, Mr. Washburn entered the service of the Chicago and Northwestern Railway Company, and was engaged on permanent way work until September, 1884, when he returned to Cornell University for a year of post-graduate work.

From July, 1885, to July, 1887, he was Engineer in Charge of Bridges of the Chicago and Northwestern Railway Company and was engaged in the design, erection, and maintenance of iron and wooden superstructures and masonry. From July to October, 1887, he served as Engineer in Charge of the Wisconsin, Milwaukee, and Northwestern Union, Madison and Peninsula Divisions, of the Chicago and Northwestern Railway Company. He resigned his position with that Company in October, 1887, and gave his attention to the introduction of railway devices of his own invention.

Mr. Washburn re-organized the Belt Line of the Union Stock Yard and Transit Company, and was then sent abroad by the Chicago and Northwestern, the Illinois Central, and several other railroad companies, to study railway economics in Europe. On his return from Europe, in 1889, he took an important part in the development of Purdy's Dam and Reservoir, and other sources of water supply for New York City. Later, he went to Chile as Consulting Engineer in connection with the production and shipment of nitrate of soda. His next important work was as Consulting Engineer in connection with the building of an inter-oceanic canal in Central America, a project which was abandoned when the United States Government undertook the construction of the Panama Canal.

After several years spent in developing coal, iron mining, and water power projects in the South, Mr. Washburn discovered the possibilities of the application of hydro-electric power to the fixation of atmospheric nitrogen for the manufacture of fertilizers. The result was the organization of the American Cyanamid Company. Factories were then erected, on the Canadian side, at Niagara Falls, which have since developed an annual production of cyanamid for fertilizer and chemical use valued at more than $10 000 000.

* Memoir compiled from information furnished by Bruce Barton, Esq., and on file at the Headquarters of the Society.

Before the outbreak of the World War, Mr. Washburn realized that if the Chilean supply of nitrate of soda, on which the United States depended for making explosives, was cut off, this country would be in a serious situation in case of war.

As early as 1910, he had considered the possibility of making the United States independent of South American nitrate deposits so that, in case of an enemy blockade, the country could produce its own nitric acid. On the entrance of the United States into the World War, the American Cyanamid Company was requested by the Ordnance Department of the Government to organize the Air Nitrates Corporation to construct and operate the huge $50 000 000 air nitrates plant at Muscle Shoals, Ala. Mr. Washburn was President of the Corporation. The armistice was declared, however, before the Muscle Shoals plant and two others near Cincinnati and Toledo, Ohio, were completed. At the time of his death, which occurred at his home in Rye, N. Y., on October 9, 1922, Mr. Washburn was Chairman of the Board of Directors of the American Cyanamid Company.

He was married in 1890 to Miss Irene Russell, of Augusta, Ga., and is survived by his widow, a son, and a daughter.

Mr. Washburn was also President of the Goodman Manufacturing Company, of Chicago, and a Trustee of Cornell and Vanderbilt Universities. He was a member of the American Institute of Mining and Metallurgical Engineers, the American Institute of Electrical Engineers, the Western Society of Engineers, and American Electro-Chemical Society. His clubs included the University, Sigma Chi, Alpha Delta Phi, and Century, of New York City, and the Apawamis Club, of Rye, N. Y.

Mr. Washburn was elected a Member of the American Society of Civil Engineers on November 7, 1888.

ROBERT ATTWELL WAY, M. Am. Soc. C. E.*

DIED APRIL 6, 1901.

Robert Attwell Way was born on January 29, 1854. In 1871, he entered the Royal Indian Engineering College at Cooper's Hill, Egham, Surrey, England, from which he was appointed as Assistant Engineer in the Public Works Department of the Government of India.

He proceeded to India in 1873 and was assigned to the Rajputana Malwa Railway. In 1879, he was transferred, as Executive Engineer, to Kathiawar, and was placed in charge of the survey and construction of the Gondal Branch of the Bhavnagar Gondal Railway, for which he received the recognition of the Government of Bombay.

In 1887, Mr. Way's services were lent to the Bengal Nagpur Railway as Superintending Engineer, and, in 1889, he was appointed Engineer-in-Chief of the construction of the Delhi-Umballa-Kalka Railway, which he completed in 1892. He then returned to the service of the Government and for the next few years was employed on surveys for lines to connect Assam and Burma.

In April, 1896, he was selected by the Board of Directors of the Assam-Bengal Railway as their Agent and Chief Engineer in succession to Mr. John Walker Buyers. In April, 1901, after five years service with the Company, he died in Chittagong, India, after a short illness.

The difficulties and privations of his work in India sowed the seeds of disease which killed Mr. Way in the prime of life. He was kindly and cheerful in disposition and his ability as an Engineer was considerable. In fact, he was regarded as one of the coming men in connection with railway development in India.

In acknowledging the news of his death, the Board of Directors of the Assam-Bengal Railway, wrote as follows:

"Mr. Way was appointed to the Agency and Chief Engineership in April, 1896, and was thus connected with the railway for a period of five critical years, covering the completion of the Chittagong Section and its organization on an open-line basis, as well as the execution of the greater part of the construction work on the Northern Sections of the line, work of an exceptionally arduous character, and sufficient by itself to tax to the full the skill and energy of those responsible for it.

"During this period, Mr. Way not only sustained both as an Engineer and a Manager the high reputation with which he entered the service of the Company but, in the opinion of the Board, greatly added to it.

"The Directors feel that they have lost an Officer who, by his ability and devotion to his work, commanded their entire confidence, and they deplore his death as a great misfortune to the Company."

Mr. Way was elected a Member of the American Society of Civil Engineers on June 7, 1893.

* Memoir prepared by Evelyn Brook-Fox, Assoc. M. Am. Soc. C. E.

EDGAR TRUE WHEELER, M. Am. Soc. C. E.*

DIED MARCH 2, 1922.

Edgar True Wheeler was born in San Francisco, Calif., on November 3, 1870. He was the son of Samuel H. Wheeler, a prominent San Francisco engineer, who originated the first cable-car system in that city. Edgar True Wheeler received his first schooling in San Francisco and, later, moved with his parents to the vicinity of Westminster, Calif. Subsequently, he moved to San Diego, Calif., where he became actively engaged in the Engineering Profession, having been attached to the City Engineer's Office from 1886 to 1889.

In 1889 and 1890, he was engaged in location work on the Cuyamaca Railroad in San Diego County, which is now the San Diego and Arizona Railroad, when he was appointed to make the preliminary survey for the Bear Valley Irrigation Company and the Arrowhead Reservoir Company.

In 1891, Mr. Wheeler joined the Engineering Department of the Seattle, Lake Shore and Eastern Railroad and was engaged on bridge construction. The lure of Southern California made itself felt, however, and, in 1892, he was again appointed Engineer for the Arrowhead Reservoir Company, in charge of the topographical and irrigation surveys.

In 1893, he entered the Engineering Department of the City of Los Angeles, and was Assistant in charge of the Outfall Sewer. On the completion of this work, he became associated with James T. Taylor, M. Am. Soc. C. E., in general engineering, until 1894, when he re-entered the City Engineering Department of Los Angeles. He remained with this Department until 1896, when he became Assistant Engineer of Construction for the Pasadena and Pacific Electric Railway.

From 1896 to 1900, Mr. Wheeler was Assistant Engineer of Bridges and Tunnels, which the City of Los Angeles was building at that time. During the construction of the Third Street Tunnel, he met with an accident, and, while recuperating, left for Mexico and joined the Engineering Staff of the Mexican-Central Railroad. In 1901, with his injury still troubling him, he went to Michigan, where he became Principal Assistant in charge of the construction of all coal docks and bridges on the Copper Range Railroad. The cold weather again sent him to Southern California, where he joined the Engineering Staff of the Pacific Electric Railway, in charge of all bridge construction and designing. During 1905 and 1906 he was engaged in private practice, but still had charge of all bridge designs for the Pacific Electric Railway.

Mr. Wheeler then entered the firm of the Mercereau Bridge and Construction Company, General Contractors, specializing in bridges, wharves, and concrete structures, as Vice-President and Engineer, in charge of all estimates, designs, and construction.

In 1911, he organized the Edgar T. Wheeler Company, a corporation engaged in general engineering and contracting business, of which he was President.

* Memoir prepared by C. W. Resmaw, Assoc. M. Am. Soc. C. E.

During the World War, he left the corporation as an active member, and entered the Government service as Superintendent of Construction at Rockwell Aviation Field. Later, he entered the Army Service, as a Captain of Engineers, and was stationed with the 125th Engineers.

In April, 1920, Mr. Wheeler was associated with the late Homer Hamlin, M. Am. Soc. C. E., former City Engineer of Los Angeles, with whom he was very intimate, in making a survey of the Colorado River, for possible dam sites and also to obtain additional data on the Boulder Canyon Project. This trip was made in rowboats, and was spoken of by Mr. Wheeler as being the best vacation he had ever had.

In January, 1921, he again revived the charter of the Edgar T. Wheeler Company, which had been inactive during the war, and managed it until his death. On February 25, 1922, he was taken ill with influenza, which, a few days later, developed into pneumonia, and caused his death on March 2, 1922, after an illness of five days.

Mr. Wheeler had made extensive plans for the future, and was very optimistic regarding the growth of Los Angeles, as well as the prosperity of the coming year; he was a great "booster", having been a pioneer of Southern California. Due to his splendid personality, he had a host of friends and acquaintances. As an engineer, he had developed wonderful judgment, and in his death the Engineering Profession has lost a valuable member.

Mr. Wheeler was a member of the American Society of Military Engineers, and of the Los Angeles Section of the Society. The latter passed the following resolution on his death:

"*Whereas,* In His infinite wisdom it has pleased Almighty Providence to take from us our beloved fellow-member Edgar True Wheeler, and,

"*Whereas,* The Los Angeles Section, American Society of Civil Engineers, desires to express its sympathy to his bereaved widow and children and condole with them in their grief, and

"*Whereas,* His removal from among us will prove an irreparable loss to the world and to his friends to whom he has been throughout his professional life so assiduously devoted, and

"*Whereas,* We desire to express our deep appreciation of his upright and manly character as a citizen and engineer. He was thorough, careful, reliable, honorable, and conscientious, and one who was never swerved from the path of rectitude by fear or favor. As a straightforward citizen, he set a noble example for the young of all classes and, particularly, the members of his chosen profession.

"*Therefore, Be it Resolved,* That these resolutions be spread upon the minutes of this Society and a copy sent to his family as our expression of sympathy and condolence in their affliction.

"'We cherish the memory of his noble character, his lofty patriotism, and the wisdom of his counsel. We, that loved him so, followed him and honored him and made him our pattern to live and to try'."

Mr. Wheeler was elected a Member of the American Society of Civil Engineers on December 7, 1904.

THOMAS DELANO WHISTLER, M. Am. Soc. C. E.*

DIED NOVEMBER 21, 1921.

Thomas Delano Whistler, the son of George William and Julia DeKay Winans Whistler, was born in Petrograd, Russia, on January 19, 1856. He was the grandson of George Washington Whistler who was noted for the construction of the railroad in Russia between Moscow and Petrograd in 1843, and who was succeeded in this work by his son, George William Whistler.

Thomas Delano Whistler received his early education in Europe and in Baltimore, Md. He was graduated as a Civil Engineer from Rensselaer Polytechnic Institute, at Troy, N. Y., in 1881, and in the same year entered the office of the late William E. Worthen, Past-President, Am. Soc. C. E., in New York City, as his Assistant.

In May, 1884, Mr. Whistler was appointed by the New York Aqueduct Commission as Engineer on Surveys, and, in January, 1885, was promoted to be Assistant Engineer on Construction, retaining this position until August, 1886, when he was appointed Resident Engineer on the Colorado Midland Railroad. On the completion of this work, Mr. Whistler returned to New York City, and, in March, 1887, was appointed Division Engineer in charge of the up-town plant of the New York Steam Company, where he remained until June, 1887.

Mr. Whistler's last years were spent at Nagold, in the Black Forest, Würtemberg, Germany, where he died on November 21, 1921. Although he had resided in Germany for a great many years, he had never relinquished his American citizenship.

He was married to Miss Sophie M. Ames, of Cambridge, Mass., who died on August 8, 1922. He is survived by his daughter, Mrs. Percy D. Haughton, of Charles River, Mass., his brother, Ross Winans Whistler, of Lenox, Mass., and a sister, Mrs. Albert Revillion, of London, England.

He was an ardent and expert fisherman and invented several devices to improve the fishing rod and reel. His exploits in killing sea bass at Newport, R. I., are well remembered.

Mr. Whistler was elected a Junior of the American Society of Civil Engineers on March 5, 1884, and a Member on May 2, 1888.

* Memoir prepared from information furnished by Ross W. Whistler, Esq., and on file at the Headquarters of the Society.

SEBASTIAN WIMMER, M. Am. Soc. C. E.*

DIED NOVEMBER 30, 1921.

Sebastian Wimmer, the son of George and Theresia (Hahn) Wimmer, was born at Thalmassing, near Ratisbon, Bavaria, Germany, on January 5, 1831. His father was a hotel proprietor at Thalmassing, but removed to Munich in 1833, where the son was graduated from the Technical and Polytechnic Schools. He also took an engineering course at the latter in 1850-51.

On June 2, 1851, Mr. Wimmer, with his uncle, the late Archabbot Boniface Wimmer, O.S.B., arrived in New York City. He went from there to Pittsburgh, Pa., where he was graduated from the Iron City College, having taken a course in bookkeeping and business methods.

He began the practice of his profession in the United States with the engineering firm of Hastings and Preisser, of Pittsburgh, Pa., with which the writer's father, the late Sigismund Low, was also associated. Messrs. Wimmer and Low were intimate friends, whose acquaintance lasted many years.

From Pittsburgh, Mr. Wimmer went to New Orleans, La., where he remained for six months, returning to Pittsburgh, in 1853. He then secured from the late W. Milnor Roberts, Past-President, Am. Soc. C. E., Chief Engineer of the Allegheny Valley Railroad, an appointment as Assistant Engineer in charge of the Division between Pittsburgh and Kittanning, a distance of 45 miles. He was engaged on this work until October, 1856, when he went to Minnesota. Soon after locating at St. Paul, he was appointed Assistant Engineer of the Faribault Division of the Minneapolis and Cedar Valley Railroad (now the Chicago, Milwaukee, and St. Paul Railroad), after locating the line from Minneapolis to Mendota, *via* Fort Snelling.

From Minnesota, Mr. Wimmer returned to Pittsburgh, where he left his family, and again went to New Orleans with the idea of locating there, but, on account of his health, he was forced to give up his plans. Returning to Pittsburgh, he accepted the position of bookkeeper at St. Vincent's Abbey, Westmoreland County, Pennsylvania, where he was instrumental in having a telegraph office and post-office established and where he served as the first Postmaster.

In 1861-63, under Thomas Seabrook, he made the survey for the Allegheny Valley Railroad from Garland to Oil City, Pa., and northward from Titusville. In June, 1863, he was transferred to St. Marys, Pa., to take charge of the Edward Miller and Milton Courtwright Contract, to complete the Philadelphia and Erie Railroad from Whetham, west of Lock Haven, to Warren, Pa., a distance of 143 miles.

This work having been completed early in 1865, Mr. Wimmer went to New York City, where, on March 6, 1865, he met Gen. S. V. Talcott, a relative of Capt. Andrew Talcott, Chief Engineer of the Imperial Mexican Railway, and secured an appointment on this railway, which, at the time, was under construction between Vera Cruz and the City of Mexico, 264 miles, exclusive of the Puebla Branch.

* Memoir prepared by Emile Low, M. Am. Soc. C. E.

Mr. Wimmer was assigned to the heavy construction work on what is known as the Maltrata Incline, the heavy grade line between Orizaba and Boca del Monte. He sailed from New York City on March 29, 1865, arriving at Vera Cruz on April 13. At that time, the railway was in operation as far west as Paso del Macho (Mule Pass), 48 miles. Leaving Vera Cruz on April 16, 1865, Mr. Wimmer reached the railroad terminus after a three hours' journey. The trip from Paso del Macho to Orizaba, 34 miles, was made by stage coach, and Mr. Wimmer arrived at the latter place on the morning of April 17. Orizaba, then a city of 22 000 inhabitants, was the headquarters of both the engineers and the contractors.

Capt. Talcott arrived from the City of Mexico on April 25, 1865, in company with Mr. William Lloyd, of London, England, who was Chief Engineer for the Contractors, Smith, Knight and Company, and on April 26, the party, with Mr. Wimmer, traveled on horseback to Maltrata. The next day they rode to Boca del Monte (mouth of the mountain), the head of the grade, returning to Maltrata the same evening. Mr. Wimmer had been placed in charge of Sections 1-7 (seven miles), comprising the very heavy work of the upper end of the Maltrata Incline, superseding Mr. Fred Simon, an English engineer.

Emperor Maximilian, on a tour of inspection of the railroad, arrived at Maltrata on April 29, 1865, and was entertained at dinner by the chief contractors, Smith, Knight, and Company. During this dinner, he was apprised of the assassination of President Lincoln, and, with a number of the engineers, left immediately for Orizaba.

On Sunday, April 30, Mr. Wimmer attended church with the Emperor, and, later, left for Maltrata, to take charge of his assignment. His first headquarters, at Boca del Monte, were in a shanty which housed from 20 to 30 mules and also served as a powder magazine. On July 20, 1865, he moved to new quarters, at the site of Wimmer Bridge, Section 5, and his first work was the taking of the regular monthly estimate. The center line had been run, but he had to make frequent changes in the alignment, introducing curves of 350-ft. radius, in order to reduce the work. One of these points was at Wimmer Bridge, another at the Twin Barrancas, where he located a tunnel through a projecting spur, in order to reduce two large embankments which would have required "millions" of cubic yards of material. In Mexico, the wet season occurs in June, July, August, and September, and during this period enormous slides gave much trouble.

Routine work kept Mr. Wimmer busily employed, the monotony being varied by frequent inspections by the engineers of the Railroad Company, as well as those of the Chief Contractors. Other incidents were guerrilla scares, the murders of sub-contractors by soldiers of the Liberal Government, earthquakes, etc. The Empress Carlotta also visited Orizaba on December 25, 1865, an entertainment being given in her honor.

Owing to financial difficulties, the railway work was considerably curtailed in the spring of 1866, and Mr. Wimmer procured a leave of absence to visit the United States, arriving at New York City on June 13. He returned to Mexico in September and proceeded at once to Maltrata. On October 23, he

was ordered to the City of Mexico to compile an estimate of the amount of work done by the contractor between Paso del Macho and Boca del Monte, 59.5 miles. This work occupied two months, and on January 3, 1867, Mr. Wimmer returned to Maltrata to take up his routine duties. Military operations between the contending forces of Emperor Maximilian and the Mexicans were in progress at this time and virtually compelled the abandonment of railroad construction.

Having received orders to proceed to London, England, in the interest and pay of the Chief Contractor, Mr. George B. Crawley, Mr. Wimmer, in the company of many of the engineers, contractors, and their families, left Orizaba on April 30, 1867, for Paso del Macho, which place was reached after many delays. Mr. Wimmer had with him such maps, profiles, documents, and papers, relating to the work, as were essential for the computation of quantities and the determination of the moneys due the contractor.

On Many 1, 1867, the party left Paso del Macho, on an open flat car. Owing to the ascending grade to Cameron, 8 miles, they had to hire Indians to push the car to the summit, and from there they dashed down grade eastward at great speed, as the brake on the car would not work properly. They passed through Soledad and up the next grade, where, fortunately, they were able to control the speed, to Tejeria, 10 miles out of Vera Cruz, where the railroad trestle over the barranca had been burned. From Tejeria, they telegraphed Gen. Benavides, at Casa Mata, in the outskirts of Vera Cruz, who sent them a car on which they reached Casa Mata.

Mr. Crawley and others called on Gen. Benavides, to present instructions from Gen. Porfirio Diaz (at the time besieging the City of Mexico) to allow the party to leave Mexico for England. The members of the party were then taken to Medellin, and on May 3, they left Medellin in Indian canoes for Boca del Rio, 9 miles down the river, arriving there safely. Owing to some mistake of Gen. Benavidas, Mr. Wimmer's name and that of Mr. John E. Weidenborner were omitted from the list of those permitted to leave Mexico, and they were not allowed to board the steamer, the *Jason*, with the party, although Mr. Wimmer had a United States passport signed by the Secretary of State.

On hearing of their plight (they were without money, except for an ounce of gold which Mr. Wimmer happened to have in his pocket), Mr. Crawley got in communication with Gen. Benavides, and, finally, a boat flying the American flag appeared at Boca del Rio, and to their great joy, was permitted to take Messrs. Wimmer and Weidenborner on board. They were then put aboard the *Jason*, but, afterward, Mr. Wimmer and Mr. Weidenborner left the *Jason* for the U. S. Steamer, *Virginia*, which sailed at once for Havana, reaching that city on May 10, 1867. Mr. Wimmer left Havana shortly afterward for New Orleans, and went from there to Philadelphia, Pa., arriving at that city on May 25, 1867.

On June 26, 1867, Mr. Wimmer left New York City for London, England, to compile the final estimate and adjust the accounts of Mr. Crawley, the General Contractor for the Imperial Mexican Railway. This work was completed on August 9, 1867, after which Mr. Wimmer visited the Paris Exposi-

tion and relatives in Bavaria, Germany, and returned to the United States in November. While at the Paris Exposition, he received a telegram from the London Railway Office, requesting him and Mr. Thomas Braniff to return to Mexico. But he declined the offer. Mr. Wimmer's name is indelibly associated with the Imperial Mexican Railway by having one of the bridges (Puente de Wimmer) on the Maltrata Incline named for him.

On his return to the United States, Mr. Wimmer received an offer from the late George B. Roberts, First Vice-President of the Pennsylvania Railroad, of a position on a line to be built by that Company from Driftwood to Dubois, Pa. (low grade), which he accepted.

From 1868 to 1874, Mr. Wimmer was in charge of the construction of the Bennetts Branch Extension of the Philadelphia and Erie Railroad, afterward known as the Low Grade Division of the Allegheny Valley Railroad, with headquarters at St. Marys, Pa. Frank M. Ashmead, M. Am. Soc. C. E., who was connected with this work in 1871-72-73, writes:

"Mr. Wimmer was pretty constantly in the field, and scarcely a week passed that he did not ride on horseback from St. Marys to Caledonia and the further twenty miles to Summit Tunnel, so that with that and the further ride on horseback of one or another of our party through the twenty miles to the St. Marys office, we had ample opportunity to become well acquainted with what I can candidly say was one of the noblest characters I have ever met in a long business life. This refers to his personal goodness of disposition. I think everybody liked Mr. Wimmer. As for his technical training and complete fitness to lead in constructive work, such as that upon which we were engaged at that time, I think we all bowed to his superior knowledge, although I believe a number of us younger recruits had been trained in technical colleges."

In 1874, Mr. Wimmer was elected to represent Elk County in the Pennsylvania State Legislature, and served for two terms, during which he was Chairman of the Railroad Committee, Secretary of the Committee on Counties and Townships, and a member of the Centennial Committee. In 1876, Mr. Wimmer was a Presidential Elector for Tilden and Hendricks.

From 1877 to 1879, he was engaged as Chief Engineer of the Pittsburgh and Lake Railroad, a line built by the so-called Economite Society, between Pittsburgh, Pa., and Youngstown, Ohio. Samuel Rea, Hon. M. Am. Soc. C. E., now President of the Pennsylvania System, served on this work under Mr. Wimmer.

From 1879 to 1882, he was the Chief Engineer of the New York and Northern Railroad, extending from High Bridge to Brewsters, N. Y., with an office in New York City. In the summer of 1882 (August and September), Mr. Wimmer went to Mexico as a representative of a New York Syndicate to examine the Mexican Central Railway then building. From late in 1882 to 1884, he served as Chief Engineer in charge of construction of the Erie and Wyoming Valley Railroad, from Hawley to Avoca, Pa. Returning to New York City, he built the Yonkers Rapid Transit Railway, reported on a line between Turner (Newburgh Junction), on the Erie Railroad, and Danbury, Conn. He also located lines in Elk and Clearfield Counties, Pennsylvania, for the Pittsburgh, Shawmut, and Northern Railroad.

In 1904, Mr. Wimmer was appointed Associate Chief Engineer of the Wabash Railway, comprising the Pittsburgh, Carnegie, and Western Railway, from Pittsburgh to the Ohio River, and the Pittsburgh, Toledo, and Western Railway, from the Ohio River to the connection of the Wheeling and Toledo Railway, in Harrison County, Ohio. This was one of the most difficult pieces of railroad construction in the United States, having seventeen tunnels, varying in length from 200 to 4 700 ft., and many iron bridges, including two exceptionally long ones over the Monongahela and Ohio Rivers. This work will ever stand as a monument to his integrity as an engineer.

In 1907, Mr. Wimmer left St. Marys, where he had maintained his residence since 1863, and went to Albany, Minn., where he resided on his farm until about 1919. He then returned to St. Vincent's Abbey, at Beatty, Pa., which had been founded by his uncle and of which he had been the first Archabbot, to spend his remaining years. His death occurred there on November 30, 1921, after weeks of patient suffering.

On February 12, 1857, Mr. Wimmer was married to Miss Lavinia Blakely, of Pittsburgh, Pa. To this union three sons were born, Dr. Sebastian J. Wimmer, Ernest J. Wimmer, an attorney, and William Wimmer. The latter two, with the wife and mother, preceded Mr. Wimmer to the grave. The surviving son, Dr. Wimmer, is a special representative for the Standard Oil Company.

Mr. Wimmer was one of those men of whom the world sees too few, and all those who came in contact with him know his real great worth.

Mr. Wimmer was elected a Member of the American Society of Civil Engineers on March 2, 1881, and at the time of his death was the oldest member of the organization.

CHARLES ALBERT LORING WRIGHT, M. Am. Soc. C. E.*

DIED AUGUST 17, 1921.

Charles Albert Loring Wright, the son of Charles M. Wright, Deputy Chief of Police of Springfield, Mass., and Martha R. Wright, was born on May 29, 1871.

He was graduated from the Springfield High School in 1889 and soon after began his engineering work with the firm of A. B. and A. H. Tower, Mill Architects and Engineers, of Holyoke, Mass., which firm designed and constructed many hydraulic power plants in addition to its work of mill designing.

After a period of service with the firm mentioned, Mr. Wright was employed as Assistant Engineer with the firm of E. E. and E. C. Davis, of Northampton, Mass., which was engaged by the Board of Water Commissioners of Springfield, Mass., to take charge of additional work connected with the water supply system of Springfield, at Ludlow, Mass.

On the completion of this work, Mr. Wright was employed by the City Engineering Department of Springfield, in general municipal engineering, assuming, at first, the duties of an Instrumentman, after which he was placed in charge of sewer construction, having had the direction of this work for several years under the City Engineer.

About 1898, Mr. Wright was placed in charge of the general study of bituminous materials for use in pavements, roadways, and sidewalks, which was continued for some years, together with the supervision and inspection of sewer construction.

With the growth of Springfield and the requirements of street traffic for new and improved street surfaces, the Street Department, in 1914, found that the time had arrived for the introduction of an asphalt plant at the City Yard, and this required the co-operation of laboratory work and methods.

Prior to this decision, Mr. Wright had shown a great interest in the study of bituminous mixtures for use in street pavements, and, by reason of his skill and practical knowledge, he was detailed to the general charge of the asphalt plant and the laboratory work. His competence in connection with this work was such that he was frequently called to other places for counsel and advice on difficult problems connected with street and road improvements.

Mr. Wright's practical knowledge of street and roadway work, coupled with his social nature, allied him strongly with the activities of the Springfield Automobile Club, of which he early became a member. After a time of general membership, the Automobile Club found that he was well qualified to serve as President of the organization, to which office he was duly elected, and he continued to serve as the active and progessive spirit of the Club until the time of his death. Much of the progessive activity in the affairs of the Automobile Club grew out of his intelligent leadership, and he was always alert for its progress and welfare.

* Memoir prepared by C. M. Slocum, M. Am. Soc. C. E.

Mr. Wright connected himself in early life with the Masonic Order, and attained the Thirty-second Degree, having membership also in the "Bela Grotto."

In 1900, Mr. Wright was married to Lillian Alberta Gordon Forbes, the youngest daughter of Dr. Henry Gordon Forbes and Hannah Elizabeth Meyer Forbes, of Chicopee Falls, Mass., and, until her death in 1916, found a true helpmeet in the refined and cultured association of his wife.

Mr. Wright's musical talent and his fine voice found expression through his membership in the choir of Christ Protestant Episcopal Church, in which he served for many years. He was also associated with the Hampden County Musical Association.

Mr. Wright was elected a Member of the American Society of Civil Engineers on October 10, 1916.

RALPH BENJAMIN ALLEN, Assoc. M. Am. Soc. C. E.*

DIED JUNE 17, 1922.

Ralph Benjamin Allen, the son of Williard Isaac Allen and Sarah Anne (Marsden) Allen, was born at Ithaca, N. Y., on August 31, 1883. He received his early education in the public schools of New York State. From 1908 to 1909 he was in the employ of the United States Government, in connection with pier and crib construction on Lake Ontario.

From 1909 to 1912, Mr. Allen was engaged as Tracer and also as Engineering Draftsman in the office of the State Engineer of New York, detailing and estimating in connection with Barge Canal bridges. From 1912 to 1914 he was connected with the Beaver Board Company, and also with the Shawmut Manufacturing Company, as Assistant Engineer in charge of design and construction, and also as Assistant to the Chief Engineer in connection with a 7 500-h. p. hydro-electric development and a steel and concrete pulp and paper mill costing $3 000 000.

From 1914 to 1915 Mr. Allen was Bridge Engineer in the office of the State Engineer of New York, in connection with the design of bridges over the Barge Canal. From 1915 to 1917 he was Assistant to the writer, and from May to August, 1917, he was at the Reserve Officers' Training Camp.

From August, 1917, to June, 1919, Mr. Allen was 1st Lieutenant with the 25th Engineer Regiment, U. S. A. He served in France from November, 1917, to May, 1919, receiving his discharge on June 2, 1919. Soon after his discharge from the Army he became connected with the Beaver Board Companies, Buffalo, N. Y., and was stationed at the plant at Anderson, Ind., as Engineer in charge of design and construction, where he remained until September, 1921. From September, 1921, until the time of his death, he was employed as Assistant to the writer.

Mr. Allen died very suddenly on June 17, 1922, at Albany, N. Y. He was married in Binghamton, N. Y., on December 20, 1906, to Marguerite May Finch, by whom he is survived.

Mr. Allen was devoted to his engineering work, in which he was intensely interested. He was of most pleasing personality and a thorough and conscientious workman. He had many devoted friends, by whom his untimely death is greatly regretted.

Mr. Allen was elected an Associate Member of the American Society of Civil Engineers on March 13, 1917.

* Memoir prepared by William Russell Davis, Assoc. M. Am. Soc. C. E.

FRANK EPHRAIM CHESLEY, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 9, 1922.

Frank Ephraim Chesley was born in Center Township, Fayette County, Iowa, on May 8, 1879. He was educated in the country schools, and was graduated from the Fayette High School in 1896. In March, 1901, he entered the State University of Iowa, and was graduated in June, 1904, with the degree of Bachelor of Science in Civil Engineering. He received the degree of Civil Engineer from the University in 1912.

From 1904 to 1908, Mr. Chesley was engaged in general engineering work as Inspector and Instrumentman on different engineering projects. From 1908 to 1910, he was employed as Resident Engineer in charge of water-works, sewage disposal plants, electric lighting plants, and paving work in various towns in Iowa. For a short period in 1912, he served as an Inspector, in the U. S. Corps of Engineers at Sterling, Ill., after which he was again engaged in municipal engineering work in Pennsylvania and elsewhere.

During subsequent years, Mr. Chesley was engaged in an engineering capacity in connection with municipal improvements and industrial works, serving as Resident Engineer in charge of the construction of rolling mills, blast furnaces, open-hearth furnaces, and other work connected with steel mills. He was also engaged for some time in connection with the U. S. Government Munitions Plant at Neville Island.

In 1919, he joined the staff of The Ludlow Engineers at Winston-Salem, N. C., doing general office work and outside practice and, later, was assigned to the position of Resident Engineer on sewer construction at Goldsboro, N. C.

Mr. Chesley was not a man of robust health, but he was very energetic, persevering, and loyal to his duty. His work as an engineer, although brief, the ability he displayed, and his intense devotion to his Profession gave abundant promise of a useful and honorable career had his life not been terminated at such an early age.

He was a member of the Masonic Fraternity. A few months before his death, he had received a certificate as Registered Engineer under the new License Law of the State of North Carolina.

On August 15, 1911, he was married to Mollie L. Livecay, of Halifax, N. C., who, with his mother, survives him.

Mr. Chesley was elected an Associate Member of the American Society of Civil Engineers on December 31, 1913.

* Memoir prepared by J. L. Ludlow, M. Am. Soc. C. E.

REGINALD GILLON CHRISTOPHERS, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 13, 1918.

Reginald Gillon Christophers was born in Dunedin, New Zealand, on August 8, 1882, and was the eldest of a family of five sons.

He began his schooling in Dunedin, but his father, who was a Bank Manager was transferred to Invercargill when his eldest son was 9 years old. At Invercargill, the boy went through the Public School course of New Zealand, and afterward spent three years, 1897-99, at the Southland Boys' High School, from which he was matriculated. While at school, he was diligent and thorough in his duties and, at the same time, took a leading part in the school sports, excelling at Rugby football.

It was while he was a student at the High School that Mr. Christophers began his military training, rising to the rank of Sergeant in the High School Cadet Corps. On leaving school, he joined the Volunteers and again succeeded in earning a Sergeant's Certificate at an unusually early age. This training, no doubt, stood him in good stead when he went into camp to prepare himself for the part he was to play in the World War.

On leaving school, Mr. Christophers took up agricultural pursuits, but after a short time, decided to become a Civil Engineer and Surveyor. To this end, he served from July, 1901, to December, 1904, as Survey Cadet under Messrs. John Spence and T. S. Miller, respectively, Civil Engineers of Invercargill, passing with credit his examination as a Surveyor in 1904 and topping the New Zealand list for that year. In January, 1905, he accepted a position with the New Zealand Government as Assistant Surveyor, in charge of a field party on road and settlement surveys. He relinquished this position in June, 1906, and came to the United States, where he remained until 1911. During this period his record reads, as follows:

From August, 1906, to February, 1907, Transitman with the Atchison, Topeka, and Santa Fé Railway Company, on maintenance of way, at Oakland, Calif.; March to September, 1907, in charge of a location party on the Monterey, Fresno, and Eastern Railway, at Monterey, Calif.; October, 1907, to January, 1908, Engineer on Construction for the Ransome Concrete Company, at San Juan, Calif., on heavy foundation work, in charge of all earthwork, reinforcement, etc., and also assisted the Superintendent; June to September, 1908, Transitman for the U. S. Deputy Surveyor, on the summit of the Sierra Nevada Mountains, California; September, 1908, to December, 1909, Transitman and Instrumentman on location and construction of the Western Pacific Railway, serving for two months as Resident Engineer on 20 miles of construction; January to April, 1910, made a contour survey of a large dam site for the Sierra and San Francisco Power Company, Angels Camp, Calif.; April to November, 1910, Assistant Engineer with the Western Pacific Railway Company in charge of a party on filing map surveys in Feather River Canyon, California; December, 1910, to March, 1911, with the Union Lumber Company, Fort Bragg,

* Memoir prepared by Denniston Cuthbertson, Esq., Invercargill, New Zealand.

Calif., making an estimate of the cost of building railways; March to August, 1911, Assistant Office Engineer to J. B. Pope, M. Am. Soc. C. E., who was in charge of revaluation for the Southern Pacific Company, making revaluation profiles and estimating the cost of bridges and culverts on the entire system of the Oregon Railway and Navigation Company; and August to December, 1911, District (Locating) Engineer for the Coos Bay and Eastern Railway in Oregon, making a preliminary survey for 45 miles of proposed railway.

Returning to New Zealand in 1911, Mr. Christophers began the private practice of his profession as a member of the firm of Robinson and Christophers, at Stratford, Taranaki, the firm acting as Consulting Engineers to Stratford Borough and other public bodies, as County Engineers, Whangamomona County, etc.

In 1915, Mr. Christophers was appointed Engineer to the Hobson County Council, and filled this position with distinction until 1917, when in order to join the New Zealand troops in the World War, he entered camp as a Second Lieutenant and was attached to the Machine Gunners. He soon gained the reputation of being a capable and thorough officer, keen about his work and loved and respected by his fellow officers and the men under him, to whose welfare he never tired of devoting himself. Lieut. Christophers was a strict disciplinarian, not only with his men, but also with himself, but his absolute fairness and impartiality soon gained for him the confidence of the troops and made his task a happy one.

He sailed from New Zealand, in June, 1918, and after a short period of training in England, joined the First Battalion, Otago Infantry Regiment, in France, from which time, he took part in some heavy fighting. He met his death in a particularly gallant manner: Leading his men in a charge against the enemy's trenches, he was held up by barb wire, and as he was cutting his way through, he was shot in the throat by a German officer. He lingered for a few days and died in the Fourth Casualty Clearing Station, France, on October 13, 1918.

Lieut. Christophers lived his short life with a fullness given to few to enjoy. He was one who, holding the highest ideals, lived up to them nobly, setting an example which any one might do well to emulate, and finally giving his life in the finest way possible, in defense of his King and country.

While in the United States, Lieut. Christophers was married to Miss Alice Mildred Vyner, who traveled from New Zealand to join him. He is survived by his wife and two sons. His father and mother also survive him, but of their family of five sons, only one remains, the other four having given their lives to their country in the World War.

> "They shall grow not old as we who are left grow old.
> Age shall not weary them nor the years condemn.
> At the going down of the sun and in the morning
> We will remember them."

Lieut. Christophers was elected an Associate Member of the American Society of Civil Engineers on June 24, 1914.

GEORGE ROBERT DAVIS, Assoc. M. Am. Soc. C. E.*

DIED MARCH 31, 1922.

George Robert Davis was born in Riverside, Calif., on October 17, 1877. At the age of 20, he joined the United States Geological Survey in the capacity of Traverseman, at Elsinore, Calif. In 1912, he was promoted to the position of Topographic Engineer in charge of the Pacific Division of the Survey, which includes the States of Washington, Oregon, California, Utah, Arizona, and the Territory of Hawaii, a position which he held until his death, at San Francisco, Calif., on March 31, 1922.

He is survived by his wife, Mrs. Adelena Marie Fontaine Davis, and a daughter.

During the years of his service with the Government, Mr. Davis surveyed much unexplored territory in the Western States, and was recognized as an expert Topographic Engineer. Included in his work of topographic surveying was the mapping of the Mount Whitney, Mount Goddard, Bakersfield, and McKittrick Quadrangles, in California, as well as mapping in the Yosemite National Park, and King's River Canyon, California, in Mt. Rainier National Park, Washington, in the Territory of Hawaii, and many other areas. Mr. Davis has thus indelibly imprinted his lifework on some thirty Government maps portraying the highest type of topographic mapping. In his death, Geographic Science has sustained a distinct loss.

Mr. Davis was a Fellow of the American Geographical Society, a member of the National Geographic Society, the Sierra Club of San Francisco, the City Club of Washington, the Sutter Club of California, and of Evergreen Lodge, F. and A. M., of Riverside, Calif. In 1915, he was a Government delegate to the International Engineering Congress in San Francisco and, in 1920, a delegate to the Pan-Pacific Scientific Congress in Honolulu, Hawaii.

Mr. Davis was elected an Associate Member of the American Society of Civil Engineers on April 3, 1907.

* Memoir prepared by H. C Rizer, Chf. Clerk, U. S. Geological Survey, Washington, D. C.

ARCHIE LEE HARRIS, Assoc. M. Am. Soc. C. E.*

DIED JULY 17, 1922.

Archie Lee Harris, the son of Lorenzo and Isabel (Combs) Harris, was born at Orange, Mass., on February 17, 1872. His early education was received in the public schools of his home town, where he taught for a number of years after his graduation from the High School. He then entered the University of Michigan, from which he was graduated in 1898, receiving the degree of Bachelor of Science in Civil Engineering.

His first position after graduation was that of Instrumentman and Draftsman with the United States Board of Engineers on deep waterway surveys. During 1900 and 1901, Mr. Harris was Draftsman on the New York State Barge Canal, at Albany, N. Y., and with the U. S. Isthmian Canal Commission, at Washington, D. C. Prior to 1903, he was also employed as Draftsman by the Russell Wheel and Foundry Company, of Detroit, Mich., the Hamilton Bridge Works, Limited, of Hamilton, Ont., Canada, and the East River Division of the Pennsylvania, New York and Long Island Railroad Company.

In May, 1903, soon after its organization as a separate bureau, Mr. Harris was appointed Assistant Engineer by the United States Reclamation Service and assigned to the Salt River Project, at Phœnix, Ariz. During the next seven years, he held the positions of Draftsman, Chief Draftsman, and Designer in the Field Office at Roosevelt, Ariz. (Roosevelt Dam), which, during the early years of construction, was the principal engineering office for the Project. During this time, he had a responsible part in the design and construction of the larger structures on the Project, but there are two, namely, the diversion dam at the head of the Roosevelt Power Canal and the Granite Reef Diversion Dam, the largest river structure below the reservoir, for the design of which he was largely responsible and is entitled to especial credit. Incidental to the design of these dams, Mr. Harris made some original investigations and studies, in an effort to improve on the lifting devices for the outlet gates which, as previously designed, were operated with great difficulty. After conducting a series of experiments on full-sized gates under actual field conditions, he found that the coefficient of friction which had formerly been used was entirely too low. By the use of new coefficients, he was able to design operating mechanisms which gave complete satisfaction, resulting in the adoption of new standards by the Reclamation Service. He thus came to be recognized as an expert in this work, and designed practically all the gates built on the Project thereafter.

About the time the Roosevelt Dam was completed, and near the completion of his period of service for the Government, Mr. Harris was the victim of an accident which nearly resulted fatally. While engaged in an inspection of the outlets and tunnel at the bottom of the dam, with water at a high elevation in the reservoir above, the gates were opened suddenly by mistake and he and

* Memoir prepared by M. R. Kays, Assoc. M. Am. Soc. C. E.

a companion were caught in a rushing torrent of water and swept through several hundred feet of tunnel into the river channel below. His companion was drowned, and Mr. Harris escaped only by the exercise of extraordinary presence of mind and great physical effort.

Mr. Harris resigned from the Reclamation Service in 1910 and opened an engineering office in Phœnix, specializing in irrigation. He designed and laid out a number of private irrigation systems and also laid out the deep-well pumping and distribution system for the 12 000-acre, long staple cotton plantation of the Goodyear Tire and Rubber Company, at Litchfield, Ariz. In 1915, he moved to Los Angeles, Calif., where he opened a consulting office, continuing, however, to serve several of his Arizona clients whose work occupied much of his time until his death. His principal work, outside of irrigation, was the design and construction of a reinforced concrete pleasure pier at Manhattan Beach, Calif. At the time of his death, Mr. Harris was Consulting Engineer for a number of proposed irrigation and power projects in Arizona, including the 40 000-acre project at Beardsley, near Phœnix, for which he had directed all the engineering work since its inception. He was also employed, in the capacity of Chief Engineer, by the Paradise-Verde Irrigation District, at Phœnix, for which he was completing final plans and estimates for a system to irrigate from 80 000 to 100 000 acres of land adjoining the present Salt River Project. As planned by him, the system was to include several large storage and diversion dams on the Verde River and other streams, one or more hydro-electric power plants, and a modern, concrete-lined, distribution system.

Mr. Harris was married on December 25, 1911, to Miss May Alderman who, with a daughter, Louise, survives him.

His untimely death cut short a most promising professional career and prevented accomplishments for which he was especially well fitted by training and years of experience and observation. Nevertheless, he had become recognized as an authority on irrigation, in the Southwest, and especially in Arizona, where he was intimately familiar with conditions. He was an enthusiastic believer in reclamation, but it was a conservative enthusiasm resulting from a thorough and conscientious study of the many matters which must be taken into account in laying the foundation for successful development. He was a builder and asked for no greater reward than to have a part in the constructive work of controlling and utilizing the wasted water resources with the resulting permanent contribution to the wealth and producing power of the nation. He was a high type of engineer, active and resourceful in the field, painstaking and thorough in investigation and analysis, conservative in his advice and recommendations, considerate of associates and subordinates, and thoroughly loyal to the interests of employer or client.

Mr. Harris' rating as an engineer was equalled only by his standing and reputation as a man. Just as he set up as a shrine and ever kept in view and observed a rigorous code of professional ethics and the highest standard of service, so did he adopt and govern himself by the highest ideals of personal conduct and citizenship and live and practice them in his daily life. Brought up

in the Universalist faith, although not in later years an active member of any denomination, he was a true Christian gentleman, because he not only believed in the rules laid down for guidance in the Good Book, but he practiced them in his work, in his dealings with his fellow men, and in his home. Although he was affable, companionable, and entertaining, with a normal sense of humor, he was given to reflection and the study of serious things, and enjoyed discussions of the political, economic, and moral questions of the day. His interest in the worth-while things, which with him were favorite topics of conversation, made him a delightful companion. Above all, however, he was a gentleman—clean, moral, honest, and upright in every respect—and he will be remembered by those who knew him well, for his genuine friendliness, his even disposition, his unselfishness, his respect for the rights and views of others, and his high-minded conception of service. "Honorable and high-minded" and "a very decided loss to the community and to the Engineering Profession", are typical of the testimonials volunteered by distinguished professional associates who knew him best.

He was the author of a paper entitled, "The Diversion of Irrigating Water from Arizona Streams", which was published by the Society.* He was a member of the American Association of Engineers.

Mr. Harris was elected an Associate Member of the American Society of Civil Engineers on May 31, 1910, and was an active member of the Los Angeles Section of the Society.

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 932.

HERMAN DAVID HIRSCH, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 30, 1922.

Herman David Hirsch was born in New York City, on October 19, 1886, and received his preliminary education in the public schools of that City. After his graduation from High School, he won a scholarship to Cornell University, and was graduated with honors from the Department of Civil Engineering in 1909.

Mr. Hirsch entered the employ of the American Bridge Company in July, 1909, at the Pencoyd Plant, as Structural Steel Draftsman, where he remained until October, 1910. He was then transferred to the Brooklyn Plant, and, in November, 1911, he was transferred to the New York Office, where he was engaged, as an Assistant Engineer, in making designs and estimates on highway and railroad bridges and miscellaneous steel structures.

Some of the most important works on which Mr. Hirsch was engaged during his career as an engineer were designs for the emergency dams for the Panama Canal, a transfer bridge for the Long Island Railroad, at Bay Ridge, N. Y., and steel boats for the United States Government at Hog Island, Philadelphia, Pa., during the World War. He also perfected the designs for the military bridges in France.

Mr. Hirsch served in the capacity of Assistant Engineer with the American Bridge Company until September 15, 1919, when he became associated with the United States Steel Products Company, and was sent to Cape Town, South Africa, as Resident Engineer, which appointment he held until the time of his death. He is survived by his widow and two daughters.

Although his career was cut short by his untimely end, Mr. Hirsch lived to fulfill his promise as a brilliant student, always discharging the duties of his various positions with great satisfaction and being held in high esteem by his associates for his professional ability. He was a man of sterling qualities and integrity of character, conscientious, upright, frank, and sincere, loving his work with singular devotion. His personality was winning and attractive, and all who came in contact with him were pleased to number him among their friends.

Mr. Hirsch was elected an Associate Member of the American Society of Civil Engineers on June 18, 1918.

* Memoir prepared by William H. Chorlton, M. Am. Soc. C. E.

ARTHUR FRANCIS HOLLAND, Assoc. M. Am. Soc. C. E.*

DIED JUNE 22, 1922.

Arthur Francis Holland, the son of Edwin and Margaret Holland, was born at Barre, Vt., on September 10, 1888. He received his early education in the public schools of Seattle, Wash., to which city his parents had moved. He afterward entered Norwich University and was graduated therefrom in 1913 with the degree of Bachelor of Science in Civil Engineering.

In July, 1913, immediately after leaving college, Mr. Holland entered the United States Engineer Department at Wheeling, W. Va., and remained in that service until September, 1915. During this time, he served as Inspector and had charge of construction work on the Ohio River locks and dams in the Wheeling District. He was also engaged on the design, estimates, drafting, and office computations and records, in connection with these structures.

In September, 1915, he entered the employ of The Koppers Company, as Instrumentman, on the by-product coke plant then being constructed for the River Furnace Company, at Cleveland, Ohio. In March, 1916, he was transferred to the plant being built for the Seaboard By-Product Coke Company, at Jersey City, N. J., as Field Engineer, in which capacity he had charge of the Engineer Corps and all layout work in connection with the plant, until August, 1917.

In September, 1917, Mr. Holland entered the Engineer Reserve Officers' Training Camp at Camp American University, Washington, D. C., and was assigned to Company 2. On the completion of the course, no openings were available in active service, and he returned from camp in November, although he was not discharged from the service until some time during 1918.

In December, 1917, he returned to the service of The Koppers Company and was placed in charge of the construction of a large by-product coke oven plant for the Providence Gas Company, at Providence, R. I. This work was completed in June, 1919. From July, 1919, to February, 1920, Mr. Holland was in charge of the tearing down and rebuilding of a battery of coke ovens for the Inland Steel Company, at Indiana Harbor, Ind. During the summer of 1920, his health did not permit him to engage in active work, although part of this time was spent in the Pittsburgh Office of The Koppers Company. In November, 1920, he was placed in charge of the work of dismantling and rebuilding a coke oven battery for the Milwaukee Coke and Gas Company, at Milwaukee, Wis., and it was while he was thus engaged that he was overcome by the illness which resulted in his death on June 22, 1922.

Mr. Holland was a man of unusual character and integrity, loyal to his associates, conscientious and painstaking in his work. His own comfort and convenience were never permitted to interfere with his duty to his employer, and he was kind and considerate to, and well beloved by, his subordinates.

* Memoir prepared by D. M. Craig, Esq., Pittsburgh, Pa.

He was made a Master Mason on June 1, 1920, and was a member of Wetzel Lodge No. 39, F. and A. M. He is survived by his wife, who was Quintie Louise Potts, and two sons, John Arthur and James Edward.

Mr. Holland was elected an Associate Member of the American Society of Civil Engineers on October 14, 1919.

WALTER LAWRENCE HULL, Assoc. M. Am. Soc. C. E.*

DIED JUNE 20, 1922.

Walter Lawrence Hull, the son of Edward Fred and Hattie Louise Hull, was born at Ossining, N. Y., on April 19, 1885. He was educated in the Public Schools of Brooklyn, N. Y., and finished his schooling at Cooper Union, New York City.

In 1902, Mr. Hull commenced the practice of his profession as a Draftsman in the City Surveyor's Office, Brooklyn, N. Y. Subsequently, for a period of about six years, he served in various engineering capacities for the Lehigh Valley Railroad Company, in New York City; the Public Service Railway Company of New Jersey, in Newark, N. J.; the New York Central and Hudson River Railroad Company, in Yonkers, N. Y.; the Pennsylvania Railroad Company, on the North River Tunnels, in New York City; with Mr. Charles Cranford, Contractor, in Brooklyn; and with the Hudson and Manhattan Railroad Company, on tunnel work, in New York City.

In 1908, Mr. Hull entered the service of the Delaware, Lackawanna and Western Railroad Company, in the Engineering Department, and continued in the employ of that Company for about eleven years, except during 1910 and 1911, which he devoted to private practice in water-proofing work. During the greater part of his service with the Delaware, Lackawanna and Western Railroad Company, Mr. Hull specialized in construction, his last work having been as Resident Engineer on the elimination of grade crossings at Passaic, N. J. Mr. Hull's ability was well adapted to construction work, which reflected his well developed esthetic side, and his originality and progressiveness often resulted in the adoption by the Company of novelties in design and methods of construction which proved to be valuable.

Mr. Hull's achievements on track elevation through Orange, N. J., for the Delaware, Lackawanna and Western Railroad Company, on which he was engaged previous to his work at Passaic, N. J., resulted in his appointment as City Engineer of Orange in 1919, which position he held at the time of his sudden death on June 20, 1922, from spinal meningitis, after an illness of only two days.

In 1906, Mr. Hull was married to Florence Irene Erwood, of Brooklyn, N. Y., who, with two sons and one daughter, survives him.

He was a member of Corinthian Lodge, F. and A. M., of Orange, N. J., the New England Society of Orange, N. J., and the American Road Builders Association.

Mr. Hull was elected an Associate Member of the American Society of Civil Engineers on January 19, 1920.

* Memoir prepared by William H. Speirs, Esq., Glen Ridge, N. J.

CLIFFORD MARSHALL KING, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 2, 1922.

Clifford Marshall King, the son of Judge and Mrs. Edmund B. King, was born in Sandusky, Ohio, on December 17, 1879.

He received his elementary education in the Sandusky schools and at Oberlin Academy. In September, 1897, he entered Adelbert College, Western Reserve University, at Cleveland, Ohio. In April, 1898, he enlisted in the Army and served as Battalion Sergeant Major of the Second Battalion, 6th Regiment, Ohio Volunteer Infantry, until September, 1898, when he was honorably discharged by order of President McKinley. Mr. King received his B. A. Degree from Western Reserve in 1901, and in the same year, entered Cornell University, from which he was graduated with the degree of Civil Engineer in 1904. He received Sigma Xi honors and was a member of the Alpha Delta Phi Fraternity.

After his graduation and until 1907, Mr. King was Assistant Engineer in the United States Reclamation Service and Deschutes Irrigation and Power Company. From January, 1908, until the latter part of 1912, he served as City Engineer of Sandusky, Ohio. After completing his terms as City Engineer, and up to the time of his taking office as County Engineer of Erie County, Ohio, in January, 1915, he devoted his time to the general practice of Civil Engineering in his own office in Sandusky.

Mr. King served as County Engineer for two years and then entered the U. S. Army as Captain in the Engineer Corps. After receiving his honorable discharge from the Army in 1919, he entered the employ of the City of Cleveland as Assistant Engineer in charge of design in the Sub-Division of Sewage Disposal, in which position he remained until his death on January 2, 1922. Just prior to his death, he had made arrangements to become associated in private engineering practice with George B. Gascoigne, M. Am. Soc. C. E., in a position similar to that which he held with the City of Cleveland.

He is survived by his widow, Edith Davis King, his parents, and one sister.

Mr. King was an ardent patriot, and in 1895 began his military training with enlistment in Company B, 16th Regiment, Ohio National Guard. In 1898, as stated, he was in the Federal Service during the Spanish-American War and when the United States entered the World War, Mr. King was among the first to offer his services. He was commissioned as Captain in the Engineer Reserve Corps on August 27, 1917, and soon after entered the Second Engineer Officers' Training Camp. After receiving his commission, he served as Assistant to the Division Engineer, 88th Division, and was detailed as Instructor of Field Fortifications at the Third Officers' Training Camp at Camp Dodge, Iowa. Soon afterward he was assigned to the 528th Engineers and was Commander of Company B from April, 1918, to March,

* Memoir prepared by George B. Gascoigne and Robert Hoffmann, Members, Am. Soc. C. E.

1919, when he was promoted to Commander of the Battalion, which office he held until the time of its demobilization at Camp Gordon, in July, 1919. He served in France from July, 1918, to June, 1919.

Mr. King thoroughly enjoyed his military experiences and had such a complete grasp of the details of the various movements in France that after his return it was most instructive and entertaining to listen to his explanations of the accomplishments of the Field Armies.

As a Civil Engineer, Mr. King has left work in and around Sandusky, which will stand to his credit for many years to come. This applies particularly to the Cedar Point Chaussée and various highways and bridges in which he specialized.

In his professional service, he was conscientious and painstaking to the last degree. His ability to apply himself to a problem showed a power of concentration possessed by few. He was an untiring worker and would never rest until the task at hand was completed.

Although quiet and reserved by nature, Mr. King nevertheless had a keen sense of humor. His politeness and chivalry were noted by every one who had occasion to meet him socially. To those who knew him intimately, he had a personal charm that made him not only highly respected but greatly beloved as well. His passing leaves a gap in the ranks of his many friends who knew him in all circumstances as a man of sterling character and high ideals.

He always kept abreast of the times and identified himself with various professional societies. He was a member of the American Association of Engineers, the Ohio Engineering Society and the American Society of Military Engineers. In addition to the professional societies, he was a member of the City Club of Cleveland and also of various Masonic bodies, in all of which he had attained a high degree of eminence.

Mr. King was elected a Junior of the American Society of Civil Engineers on March 6, 1906, and an Associate Member on July 1, 1909.

LOUIS WILLIAM KLINGNER, Assoc. M. Am. Soc. C. E.*

DIED APRIL 2, 1922.

Louis William Klingner, the son of Dr. Louis M. Klingner, was born at Toronto, Ont., Canada, on June 10, 1886. He received his engineering education at the University of Toronto from which he was graduated in 1907. This was followed by post-graduate work at Columbia University, New York City, during 1909-10.

From June to October, 1905, Mr. Klingner served as Chainman and Rodman with the Canadian Pacific Railway Company, and from May to October, 1906, he was employed as Draftsman in the office of Division Engineer F. S. Darling, M. Am. Soc. C. E., in Toronto.

Mr. Klingner was Field Draftsman and Instrumentman on the Georgian Bay and Seaboard Branch of the Canadian Pacific Railway from May, 1907, to January, 1909. While in this position, he prepared plans for the layout of docks and quays, and the harbor at Port McNichol, on the western terminal of the line. He was also engaged on the layout of the yards and the first unit of grain elevators.

From January to May, 1909, he was employed on the preparation of plans for the Toronto-Sudbury Branch, and the Georgian Bay and Seaboard Railway, of the Canadian Pacific System, in the offices of the Division Engineer.

From July to November, 1909, Mr. Klingner was with G. A. Just and Company, in Long Island City, N. Y., as Detailer on structural steel, and from November, 1909, to July, 1910, he was employed by the New York Central Railroad Company as Designer on masonry and reinforced concrete for the Grand Central Terminal, New York City.

From July, 1910, to February, 1911, he was with the Foundation Company, Limited, of Montreal, Que., Canada, in the following capacities: Superintendent of Construction on reinforced concrete oil tanks, built for the Queen City Oil Company at Toronto, from July to December, 1910, and as Assistant Superintendent on the Dominion Express Building foundations in Montreal, from December, 1910, to February, 1911.

From February, 1911, to March, 1912, Mr. Klingner served as Resident Engineer on double track from Smiths Falls, west, to Glen Tay, Ont., in charge of all grading, bridges, culverts, etc., for the Canadian Pacific Railway, and from March to May, 1912, he was in charge of a party revising location on the Campbellford, Lake Ontario, and Western Railway, a branch of the Canadian Pacific Railway. In May, 1912, he was appointed Resident Engineer on Section No. 3 of the same road, in charge of all grading, laying out of bridges and culverts, and preparing plans for bridges, trestles, etc., which position he retained until August, 1913.

During the latter part of 1913 and in 1914, Mr. Klingner was employed as Company Engineer with the Dominion Construction Company, at Toronto. At the outbreak of the World War, he enlisted as a Lieutenant with the 2d

* Memoir compiled from information supplied by Fraser S. Keith, Secretary, Eng. Inst. of Canada, and on file at the Headquarters of the Society.

Field Company, Canadian Engineers, and was engaged at first in instructing and training infantry in field works and military engineering at Toronto.

From April to December, 1915, he was in charge of construction work at Niagara Camp, and from December, 1915, to March, 1916, he was with O. C. Depot Company at the Engineers' Training Depot, at Ottawa, Ont. From March to July, 1916, he served with the 8th and 10th Field Companies, Canadian Engineers, in England, and from August, 1916, to May, 1918, he was Lieutenant in the 10th Field Company, Canadian Engineers, in France. From May to August, 1918, he served as Captain of the 10th Battalion, Canadian Engineers, and from August, 1918, to July, 1919, as Staff Captain, 4th Brigade, Canadian Engineers.

Captain Klingner was awarded the Military Cross on November 16, 1916, for exceptional courage in bringing in, under enemy fire, a wounded Canadian officer at the Somme, risking his life for a comparative stranger. He was demobilized on July 6, 1919.

On his return to Canada, he accepted the position of Company Engineer and Comptroller of the International Corporation of Canada, at Montreal, and, in 1920, he was appointed District Irrigation Officer in charge of all irrigation surveys in Mesopotamia, with the Irrigation Directorate, Bagdad, Mesopotamia, having under his control eighty-four local officers, with parties in the field.

On April 2, 1922, while making a trip along the Tigris River from Bagdad to his Serrai headquarters, his motor launch was swept against a bridge pier with such force that it was crushed and sunk within 20 sec. and Capt. Klingner was drowned.

He was a man of generous nature, sterling character, and high ideals, and his loss is irreparable.

Captain Klingner was elected a Junior of the American Society of Civil Engineers on October 31, 1911, and an Associate Member on June 24, 1914. He was also a Member of the Engineering Institute of Canada.

WILLIAM ARTHUR LAFLER, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 19, 1922.

William Arthur Lafler, the second son of William Arthur and Ida Anderson Lafler, was born at Gaines, N. Y., on October 22, 1879. He received his college education at the University of Michigan, from which he was graduated in 1903 with the degree of Bachelor of Science in Civil Engineering.

In July, 1903, Mr. Lafler became Junior Engineer in the United States Engineer Department, on surveys, maps, estimates, etc., for a deep waterway from Chicago, Ill., to St. Louis, Mo., and remained on this work until March, 1906, when he went to Monterey, Mexico, as Designing and Constructing Engineer for water-works and sewers. In September, 1906, he entered the Department of the New York State Engineer on Barge Canal work, in charge of surveys and estimates of cost in the vicinity of Rome, N. Y., and, later, as Engineer in Charge of Barge Canal Contract No. 42, in the vicinity of Utica, N. Y.

After leaving the New York State Engineer's Department, Mr. Lafler opened an office in Rochester, N. Y., and did a great deal of work along hydraulic lines. At the time of his death, he was particularly interested in the reclamation of various parts of the Tonawanda Swamp in the western part of New York State.

Mr. Lafler was a man of sterling integrity of character and a high-class engineer. Those of his friends who knew him best were convinced that, had he been granted the usual span of life, he would have made his mark in the engineering field.

He was married on June 11, 1919, to Miss D. Virginia Heath who, with one daughter, Lois Virginia, survives him. He is also survived by his mother and two brothers.

He was a member of Oriental Lodge No. 224, F. and A. M., of Utica, N. Y., and of the Ad. Club of Rochester, N. Y.

Mr. Lafler was elected an Associate Member of the American Society of Civil Engineers on May 4, 1909. He was elected a Member on January 18, 1922, the day before his death, but failed to qualify.

* Memoir prepared by Fred J. Wagner, M. Am. Soc. C. E.

PERCY MAPES LAU, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 13, 1922.

Percy Mapes Lau was born at Ortonville, Oakland County, Mich., on December 15, 1883.

He was educated in the public schools of Detroit, Mich., and at Pierce Business College, Philadelphia, Pa. He also studied Civil Engineering at the University of Michigan from September, 1904, to June, 1907. During 1903, Mr. Lau was employed as a Stenographer for the International Mercantile Marine Company of Philadelphia.

From September, 1907, to December, 1909, he held field and office positions on preliminary location and construction work with the San Diego and Arizona Railroad of San Diego, Calif. From December, 1909, to May, 1910, he was employed as Levelman and Draftsman on highway work with the San Diego County Highway Commission.

From May, 1910, to January, 1911, Mr. Lau served as Locating Engineer with the Detroit, Lansing and Grand Rapids Railway Company, of Detroit, Mich., and from January, 1911, to February, 1912, he was employed as Inspector and Bridge Engineer on highway construction with the San Joaquin County Highway Commission, at Stockton, Calif.

In February, 1912, Mr. Lau was appointed Chief Draftsman and Resident Engineer of Division II of the California Highway Commission, and held this position until May, 1915. During this time, for a period of six months, he was on temporary leave of absence, acting as Chief Draftsman and Field Engineer for the San Mateo County Highway Commission, of Redwood City, Calif. From May to October, 1915, he also served as City Engineer of Dunsmuir, Calif., during the construction of the concrete State highway through the corporate limits of the city.

In October, 1915, Mr. Lau was appointed Chief Engineer of the Detroit Construction Company, Limited, in charge of the location and construction of the Detroit, Pontiac and Owosso Electric Railway, extending from Detroit to Owosso, Mich., a distance of fifty-four miles. It became necessary to abandon this work owing to conditions prevailing at the time of the World War.

From 1917 to 1919, Mr. Lau was in charge of highway construction approximating $1 000 000 per year in Oakland County, Michigan, and from 1919 to 1922, he was engaged in the contracting business, during which time he constructed reinforced concrete bridges and approximately forty miles of concrete and gravel highways in Oakland and the neighboring counties.

He was killed in an automobile accident which occurred just north of Pontiac, Mich., on September 13, 1922.

Mr. Lau was elected an Associate Member of the American Society of Civil Engineers on September 12, 1916.

* Memoir compiled from information supplied by N. W. Strait, Assoc. M. Am. Soc. C. E., and on file at the Headquarters of the Society.

EMERET CLAUDE NEUDECKER, Assoc. M. Am. Soc. C. E.*

DIED JUNE 8, 1921.

Emeret Claude Neudecker, the son of Frank and Lavina (Powers) Neudecker, was born at Madison, N. Y., on June 8, 1885. His early education was obtained at Oriskany Falls Union, New York, and at Colgate Academy, from which he was graduated in June, 1906. He received his technical education at the Clarkson College of Technology, from which he was graduated in the Class of 1911.

From July, 1911, to February, 1918, Mr. Neudecker was with the New York State Engineer at Albany, N. Y. Starting as Chainman, he was promoted through the grades of Rodman, Leveler, Timekeeper, Transitman, Inspector, and Junior Engineer, and on April 1, 1917, reached the grade of Junior Assistant Engineer.

From March 1 to December 22, 1918, he was employed as Chief of Party for the Mellon-Stuart Company, his work consisting of giving grades and locations for the construction of a permanent hospital unit at Edgewood Arsenal, Edgewood, Md.

From December 22, 1918, to March 22, 1920, Mr. Neudecker was with the Shipbuilding Realty Corporation, at Newport News, Va., as Chief of Party. As such, his work consisted of giving grades and locations for all main sewers, sewage disposal plant, storm drains, walks, curbs, and gutters for the Hilton Village Housing Project. Later, he assisted in making the final record maps of this work.

Mr. Neudecker began his work for the Iowa Highway Commission on June 1, 1920, as Instrumentman on surveys. He continued with the Commission on various work, until his death on June 8, 1921. On December 9, 1920, he was transferred to District Headquarters at Storm Lake, Iowa, where he was engaged in preparing plans for road construction for the State. On April 1, 1921, he was transferred to Humboldt, Iowa, as Resident Engineer on grading and paving, and he was located at Humboldt at the time of his death.

On July 15, 1913, Mr. Neudecker was married to Sarah E., daughter of Willard Bruce and Lephe L. Sprague, of Canton, N. Y., who died in May, 1918. A daughter, Sarah Frances, survives him.

Mr. Neudecker was a man of the conservative type. He was one of the few men who actually have more ability than they themselves realize. While in Iowa, he established a reputation as a careful, accurate engineer, in recognition of which he was given the important assignment at Humboldt. He numbered among his friends all his associates in the highway work, and his sudden death was a shock to his friends in Iowa and his relatives in New York State.

Mr. Neudecker was elected an Associate Member of the American Society of Civil Engineers on April 19, 1920.

* Memoir prepared by J. F. Reynolds, Assoc. M. Am. Soc. C. E.

SPENCER BAIRD NEWBERRY, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 28, 1922.

Spencer Baird Newberry, the third son of Professor John Strong Newberry, sometime State Geologist of Ohio and, for many years, Professor of Geology at Columbia University, New York City, was born in Cleveland, Ohio, on May 11, 1857, and was graduated at the School of Mines, Columbia University, in 1878, taking his doctor's degree there one year later. He studied in Berlin, Germany, and Paris, France, for two years, and, from 1881 to 1892, was Acting Professor of Chemistry at Cornell University, Ithaca, N. Y. In 1889, he was sent by the Government as Expert Commissioner to the Paris Exposition, and, in 1893, he was a member of the Jury of Awards at the World's Fair at Chicago, Ill.

While at Cornell University, Dr. Newberry became interested in the manufacture of Portland cement, then in its infancy in the United States. Recognizing the possibilities for the future development of the cement industry, he left Cornell and founded the Sandusky Portland Cement Company with its first plant at Bay Bridge, near Sandusky, Ohio, built in 1892. He was General Manager of this Company until 1912, when he became President, which position he occupied until his death. The business developed rapidly and other plants were built in Syracuse, Ind., Dixon, Ill., and York, Pa., which have been successfully operated for many years. During the last year of his life, Dr. Newberry's energies were devoted to the planning and erection of a new plant near Toledo, Ohio, on a valuable and unique deposit of limestone discovered through a paragraph in his father's first report on the Geological Survey of Ohio in 1874, and the plant, incorporating as it does the results of his wide experience in the art, is expected to be the last word in modern cement practice. Although he did not live to see it in operation, he took the greatest satisfaction in its construction.

Dr. Newberry was the first to demonstrate that white Portland cement could be made successfully, and the factory at York, Pa., is devoted to the manufacture of this product alone, which has found wide use in ornamental architectural work. He was also successful in inventing and manufacturing an integral water-proofing compound for concrete, and had taken out many patents on chemical processes in the cement and other industries. His early investigations into the chemical constitution of cement and the determination of a theoretical formula for proportioning the raw materials have been recognized as having marked an epoch in the industry, and he was conceded to be one of the foremost authorities on the chemistry of cement as well as on the mechanical processes of its manufacture. He was also the author of many articles and monographs on the art, which are considered highly important and valuable contributions to the knowledge of this industry.

* Memoir prepared by W. B. Newberry, Esq., Cleveland, Ohio.

Dr. Newberry was married at Ithaca, N. Y., in 1882, to Clara, daughter of the Hon. Andrew D. White, then President of Cornell University. She died in 1907, leaving two sons, Andrew W. and Arthur C. Newberry, both of whom survive their father. In 1908, he was married to Helene Printy, of Vickery, Ohio, who also survives him.

Dr. Newberry was elected an Associate Member of the American Society of Civil Engineers on January 6, 1897.

ABRAHAM JOHN RUTH, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 3, 1921.

Abraham John Ruth, the son of Daniel and Anna Ruth, was born in St. Clair County, Illinois, on April 6, 1874. He was educated at the University of Kansas, from which he was graduated in 1901, with the degree of Bachelor of Science in Civil Engineering.

During his summer vacation in 1900, Mr. Ruth served as Topographer on a railway survey in Arkansas. After his graduation, in June, 1901, he entered the employ of the American Bridge Company, at Trenton, N. J., as Draftsman on the detailing of steelwork, but he resigned in April, 1902, to accept a position as Draftsman in the Bridge and Building Department of the Chicago, Milwaukee, and St. Paul Railway Company, at Chicago, Ill. In this capacity, he was engaged on the design and detailing of steel and masonry work, having had charge for one year of masonry design for the Department.

In August, 1904, Mr. Ruth went to the Pacific Coast, and entered the service of the Southern Pacific Company, having been engaged on maintenance-of-way and office work on the Los Angeles Division.

In July, 1905, he was appointed Assistant Engineer and Chief Draftsman in the Railway Department of the Cananea (Mexico) Consolidated Copper Company, and was engaged on the design of bridges, structures, machines, etc., and also assisted in superintending their construction. In February, 1911, he was also given charge of the Mechanical Drafting Room of the Company, and from June, 1912, to 1913, he served as Chief Engineer in charge of engineering and construction work. In 1913, Mr. Ruth entered the service of the Arizona Copper Company, with which Company he remained for several years. He was also engaged in the automobile business for a number of years.

On January 1, 1921, he went to Tampico, Tamps., Mexico, in the employ of the Huasteca Petroleum Company, with which Company he served until his death on November 3, 1921.

He was married on December 25, 1902, to Emma C. Eymann, who, with three daughters and one son, survives him. He is also survived by his father, eight brothers, and three sisters.

Mr. Ruth was elected an Associate Member of the American Society of Civil Engineers on March 4, 1913. He had also been a member of the Western Society of Engineers.

* Memoir prepared from information furnished by M. H. Willis, Assoc. M. Am. Soc. C. E., and on file at the Headquarters of the Society.

HUGO JULIUS SCHEUERMANN, Assoc. M. Am. Soc. C. E.*

DIED JULY 20, 1921.

Hugo Julius Scheuermann was born at Suvalki, Russia, on February 11, 1873. After completing his elementary education, he attended High School from 1885 to 1889, taking a preparatory course for Engineering Science at the Standard Government School, at Petrograd, from which he was graduated in 1892, with the degree of Civil Engineer.

After his graduation, Mr. Scheuermann was appointed Assistant Engineer with the Department of Railroads, Petrograd, with headquarters at Bialystock. He came to the United States early in 1894, and, for two years, was employed with the Southern Pacific Railroad Company on track relocation.

From 1896 to 1904, he was active in the design and construction of structural steel with the American Bridge Company, Hitchens and Company, and Milliken Brothers. In 1904, he was appointed Bridge Designer in the office of the State Engineer and Surveyor of New York State, and held this position until the time of his death, at his home in Albany, N. Y., on July 20, 1921.

Mr. Scheuermann became a citizen of the United States and married here. He is survived by three children, his wife having died several years ago.

Of a quiet and unassuming personality, he was a conscientious and thorough workman, and was devoted to his family.

Mr. Scheuermann was elected an Associate Member of the American Society of Civil Engineers on December 7, 1904.

* Memoir prepared by William Russell Davis, Assoc. M. Am. Soc. C. E.

JOHN EARL SHOEMAKER, Assoc. M. Am. Soc. C. E.*

DIED JUNE 22, 1922.

John Earl Shoemaker was born in Charleston, Ill., on September 26, 1881. He was educated at the University of Illinois, was graduated with the highest honors of his class, and received his Bachelor of Arts degree in 1903.

His first engineering work was with the Wisconsin Bridge and Iron Company. From 1905 to 1907, he was connected with the U. S. Reclamation Service on various projects. From 1907 to 1909, he was with the Departamento de Obras Publicas, Cuba, and had charge of designing and constructing several large water-supply systems. From 1909 to 1912, he was employed as Designing and Construction Engineer for various irrigation projects in Colorado, Wyoming, and Montana.

Mr. Shoemaker came to Seattle, Wash., in November, 1912, and in January, 1913, accepted the position of Assistant Engineer with the Port of Seattle in charge of outside construction. In April, 1915, he was made Assistant Chief Engineer, and remained in that capacity until March, 1916, at which time he resigned to form a partnership with Peter Swensson, under the firm name of Swensson and Company, General Contractors. This partnership lasted six years, during which time work on many important contracts was done. He occupied the positions of President and Secretary, respectively, of the Seattle Master Builders' Association.

In the spring of 1922, after refusing a responsible position with the Asia Development Company, of Shanghai, China, Mr. Shoemaker accepted a position as Chief Engineer of a large coal mining company at Herrin, Ill. He was the first man killed in the cowardly and disgraceful riot at that place on June 22, 1922, while defending the Superintendent of the mine, a cripple, who was being mistreated by the strikers. He is survived by his widow and three small sons.

Mr. Shoemaker was a man of sterling qualities and charming disposition, and commanded the respect and esteem of all who knew him. He was an able engineer, noted for his tact and ability in handling men, and was an expert mathematician.

Mr. Shoemaker was elected an Associate Member of the American Society of Civil Engineers on June 6, 1911.

* Memoir prepared by G. F. Nicholson, M. Am. Soc. C. E.

HILMAR FREDERICK SMITH, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 6, 1923.

Hilmar Frederick Smith was born at New Brunswick, N. J., on January 13, 1892, his parents being the late Dr. John B. Smith, formerly Professor of Entomology in Rutgers College and State Entomologist of New Jersey, and Marie von Meske Smith. He was educated at Rutgers College, New Brunswick, N. J., receiving the degree of Bachelor of Science in 1913, the degree of Electrical Engineer in 1920, and the degree of Civil Engineer in 1922.

Mr. Smith first began practicing his profession as a Student Apprentice with the General Electric Company, at Schenectady, N. Y., with which company he was engaged on the testing of electrical apparatus from September, 1913, to March, 1915. From March, 1915, to July, 1916, he was engaged as Assistant Engineer and later as Construction Superintendent with the Nixon Nitration Works, Millville, N. J., in connection with the construction of a $1 000 000 guncotton plant. From July, 1916, to March, 1917, he was employed by Somerset County, New Jersey, as a Surveyor and Road Superintendent on macadam road construction. From March, 1917, to April, 1918, he was employed by the Pennsylvania Railroad on the New York Division as Transitman, on yard construction work at Princeton, N. J.

He entered the Government service in April, 1918, at U. S. Nitrate Plant No. 2, Muscle Shoals, Ala., holding at that plant the position of Inspector of Construction and Mechanical Superintendent, also acting as Technical Assistant to the Commanding Officer. In July, 1920, he was transferred from the Ordnance Department to the Construction Service, Quartermaster Corps, as Assistant Supervising Engineer, in charge of the Waco Quarry Project, Russellville, Ala., during which assignment he was in responsible charge of work amounting to $535 000. In September, 1921, he was transferred to Scott Field, Belleville, Ill., as Supervising Engineer for the Construction Service of the Quartermaster Corps of the Army, at which station he was in responsible charge of the engineering and inspection incident to the construction of a $1 300 000 airship hangar.

The work entrusted to Mr. Smith's supervision during his service with the Government was of considerably greater responsibility than that usually assigned to a man of his age, but his unusual ability along both theoretical and practical engineering lines, coupled with his exceptional devotion to his profession, enabled him to discharge his duties at all times in a manner highly satisfactory, and such as would have been a credit to a much older man than himself. As a man he possessed none but the most sterling characteristics, and was universally respected and admired by all his associates. His death is a distinct loss to the Engineering Profession, in which he was taking a foremost place for a man of his years, not only by carrying on successfully the important work on which he was engaged, but as a frequent contributor to the technical press.

Mr. Smith was elected an Associate Member of the American Society of Civil Engineers on November 25, 1918.

* Memoir prepared by B. F. Vandervoort, Assoc. M. Am. Soc. C. E.

EDMUND ABIEL THORNTON, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 9, 1922.

Edmund Abiel Thornton, the only son of Thomas M. and the late Flora (Thorne) Thornton, was born at Stoneham, Mass., on September 13, 1883. While still a boy in the Grammar School, the long and serious illness of his father brought him early responsibility and served to develop the strong traits of industry and perseverance that persisted in his character during the remainder of his life. In these earlier school days, he ranked high as a scholar, and, to an unusual degree, sought to apply the knowledge gained from his studies in the problems of his daily life. He was graduated from the Stoneham High School in 1903 as Valedictorian of his class.

After his graduation from High School, Mr. Thornton passed the entrance examinations for the Massachusetts Institute of Technology, winning a scholarship. He selected the course in Civil Engineering, and close application to his studies won him other scholarships which, together with field work in his profession during summer vacations, enabled him to pay his expenses through the four years required for the completion of the course. He was graduated in 1907 with the degree of Bachelor of Science.

In July, 1907, Mr. Thornton entered the employ of the Randolph Lines (Southern Pacific of Mexico), as Assistant to the Office Engineer in Tucson, Ariz. He remained in this position until April, 1909, when he resigned to enter the employ of the Ray Consolidated Copper Company at Ray, Ariz., as Draftsman.

When he had been for about one year with the Ray Consolidated Copper Company, he was appointed Engineer on development work at No. 2 Mine, which is now the principal producing unit of the Company. While holding this position, Mr. Thornton made many important surveys, and detailed the timbering and track work for all the main haulage drifts in the mine. The exceptional accuracy and keen perception which he displayed in attacking the many difficult problems that came to him for solution during this period, could not fail to win the appreciation of his superiors, and, in January, 1914, he was placed in charge of all the surface engineering at the mines as well as of maintenance of way of the Ray and Gila Valley Railroad.

During 1914 and 1915, he planned and carried out numerous important surface improvements which the expanding plant and the rapidly growing community made necessary. It was, also, during this period, that he made preliminary surveys and studies for the flood control and diversion of Mineral Creek, and the result of these studies was an effective solution of what had previously been a serious problem to both the mines and the railroad.

On May 1, 1916, Mr. Thornton was appointed to the position of Chief Engineer, and this well-deserved promotion gave him broader scope in which to demonstrate his unusual ability. At that time, the Ray Consolidated Copper Company, as well as all the copper mines, was forcing production to the utmost in order to meet the tremendous demand for copper created by the

* Memoir prepared by Moses Brown, Jr., Esq., Ray, Ariz.

World War. With the entrance of the United States into the conflict, came the necessity of still further increase in production at the time when the draft was taking away from the working force many of the trained men who could not easily be spared. That the Ray Consolidated Copper Company was able to meet this emergency successfully, was due, in no small measure, to the efficient manner in which Mr. Thornton directed the work of his Department. His fitness for a still higher position was recognized and, on February 1, 1918, he was appointed Assistant Superintendent of Mines, which position he still held when his career was brought to an untimely end on January 9, 1922.

During the last year of his life, Mr. Thornton suffered from an illness which he bore with great fortitude and which, with characteristic bravery, he concealed even from his most intimate associates. Without doubt, persistent application to his work at this time had much to do with hastening his death. In July, 1921, he returned to his old home in Stoneham, Mass., in the hope that a long rest under skillful medical treatment would restore him to health, but, unfortunately, this object was not realized. It was typical of the man that, up to the last day of his life, he spent much of his time in study.

In 1911, Mr. Thornton was married to Miss Louise J. Parkhurst, who, with a son, William Randolph Thornton, and his father and sister, survives him.

As an engineer, Mr. Thornton was a man of exceptional ability, and a close student of all that pertained to his profession. He was far-seeing, yet practical. In everything he attempted, he showed himself possessed of an energy and power of will which could not fail of ultimate achievement.

As a man among men, he was modest and unassuming, but, whenever occasion required, he was a fearless, straightforward fighter for the cause of justice and right. Clean-minded and honorable in all things, devoted to his family, loyal to his friends and employers, he was loved and respected by all who knew him.

Mr. Thornton was elected an Associate Member of the American Society of Civil Engineers on November 4, 1914.

WINSLOW BARNES WATSON, Assoc. M. Am. Soc. C. E.*

DIED AUGUST 6, 1922.

Winslow Barnes Watson, the eldest son of the late Judge Winslow Charles Watson and Ella Barnes Watson, was born in Plattsburg, N. Y., on August 28, 1880. He was educated in the Plattsburg schools, having been graduated from the High School in 1898. It was the desire of Judge Watson that his son should be a lawyer. For that reason, he studied law in his father's office during two years following his graduation from High School. In the fall of 1900, he entered Union College, from which he was graduated in 1904 with the degree of Bachelor of Engineering.

Immediately after finishing college in 1904, Mr. Watson was appointed a Computer and Instrumentman on the construction of the New York State Barge Canal. He remained in the employ of the State of New York until 1907, when he went to Panama where he was engaged on the construction of the Canal, and where he remained until 1908. At this stage of the construction of the Panama Canal, the sanitary conditions were not all that could be desired, and, in 1908, he again returned to the construction of the New York State Barge Canal.

From 1909 to 1912, Mr. Watson was in charge of Barge Canal Construction Contract No. 25, which represented one of the largest contracts in the State, and covered a distance of 13 miles of prism excavation, one lock, three dams, six bridges, and many miscellaneous structures, aproximating in cost nearly $2 000 000. In 1913 and 1914, he was in charge of all final estimates and final accounts for the work under contract on the Barge Canal, covering the entire Eastern Division. Also, at this time, the surveys, designs, and estimates for the proposed Barge Canal Extension of the Glens Falls Feeder on the Champlain Canal were under his supervision, as well as practically all the miscellaneous surveys, designs, and constructions in the State Engineer's Department.

In 1915, Mr. Watson became associated with O'Connor and Chapman, Consulting Engineers, of Albany, N. Y. He remained with this firm until 1917, when he accepted the position of Engineer for the Holler, LaDu Corporation, of Fort Edward, N. Y.

Before the troops were called into service for the expedition against Mexico, in 1916, he had joined the 10th Regiment of the New York National Guard, and he went with that regiment as a Second Lieutenant. When the National Guard was mustered into Federal Service in 1917, as a component of the 106th Infantry, he was commissioned as First Lieutenant. He won his captaincy in France. That his military record in the World War was one to be proud of, can be easily seen by the following brief résumé: East Poperinghe Line, July 9 to August 20, 1918; Dickebusche Sector, August 30, 1918; LaSalle River, October 17, 1918; DeMar Ridge, October 18, 1918; and St. Maurice River,

* Memoir prepared by E. W. Wendell, Assoc. M. Am. Soc. C. E.

October 19, 1918, which locations, and those dates, bring to the mind the fiercest fighting.

Captain Watson's service chevrons were authorized on November 23, 1918, in Special Order 329, Headquarters 106th Infantry. His wound chevron was authorized on December 11, 1918, by S. O. 106, Hdq. 106th Inf., Special Order No. 31, which reads as follows:

"Hdq. 27th Div. U. S. A.
"American E. F. France.
"Jan. 31, 1919.

"CAPT. WINSLOW B. WATSON,
"106 Infantry.
"Citation:

"For courage and determination in making personal reconnaissance under heavy enemy fire; this near St. Maurice River, October 20, 1918."

On his return from France, and as soon as the 27th Division was mustered out of service, Captain Watson again associated himself with the Holler, LaDu Corporation, at Fort Edward, N. Y., and remained with this Corporation until 1920.

In 1920 and 1921, he had charge of a number of special departmental constructions in the Department of Public Works of the State of New York. In 1922, he was placed in charge of the construction of the new water supply project for the City of Plattsburg, in the capacity of Superintendent in Charge and Manager for Construction, and served in this capacity until his accidental death from drowning on August 6, 1922.

Captain Watson's engineering experience extended over such a wide variety of work that his knowledge of construction was extensive, and he had just reached an age when all this experience and knowledge could be utilized to its fullest extent.

He is survived by his wife, who before their marriage in 1907 was Miss Irene Signor, his daughters, Mary and Emily, his mother, Mrs. W. C. Watson, a sister, Miss Ellen Watson, and two brothers, Richard P. Watson and Mark S. Watson.

Captain Watson was elected an Associate Member of the American Society of Civil Engineers September 12, 1916.

JOHN ALEXANDER DAILEY, Affiliate, Am. Soc. C. E.*

DIED FEBRUARY 9, 1922.

John Alexander Dailey, the son of Nicholas A. Dailey, was born in Kensington, London, England, on August 13, 1839. His parents came to the United States when he was nine years old, and settled in Covington, Ky., where he was educated.

At the outbreak of the Civil War, Mr. Dailey who was located at Evansville, Ind., enlisted (April 19, 1861) in Company E, 14th Indiana Infantry. He served with this regiment for 3 years and 3 months, and fought in the battles at Green Bar, Second Bull Run, Winchester, Antietam, and Chancellorsville. He was taken prisoner by the Confederates, but was afterward paroled and honorably discharged in 1864.

From 1865 to 1872, Mr. Dailey was employed with Mr. R. C. Phillips, Surveyor and Civil Engineer, of Cincinnati, Ohio, and from 1872 to 1880, he was engaged in the Engineering Department of the Cincinnati Southern Railway Company.

Mr. Dailey then went West, and from April, 1880, until his retirement in 1904, he was connected with the Atchison, Topeka and Santa Fé Railway Company in various capacities. During the early construction of the Company's lines, he served under the late A. A. Robinson, M. Am. Soc. C. E., then Chief Engineer, at Pueblo, Colo., and at Las Vegas, N. Mex. In October, 1881, he was appointed Office Engineer at Topeka, Kans., and afterward was engaged with the late Lewis Kingman, M. Am. Soc. C. E., on the construction of the Chicago, Kansas and Western and the Southern Kansas Lines of the Company. Mr. Dailey then served as Assistant Engineer on Maintenance at various places in Colorado, Missouri, and Illinois, until 1887, when he was transferred to the office of the Chief Engineer at Topeka, where he remained until May 15, 1904, when he was given a leave of absence on account of ill health.

He was the first person to receive a pension from the Atchison, Topeka and Santa Fé Railway Company on account of faithful and efficient service. He had previously declined a pension from the Government for which he had applied, in order to refute the allusion that possibly his views regarding pensions, that an able-bodied man should not draw a pension, were prompted by a non-pensionable record. He refused to acknowledge that he was disabled as long as he was earning a living and claimed that the chronic rheumatism which had incapacitated him—contracted while in the service—was a badge of honor.

Mr. Dailey was a man of strong individuality and unimpeachable integrity, hospitable, and kindly. In Topeka, he had been familiarly known as "Diagonal Dailey", due to the fact that he had forced the State to construct diagonal walks through the State House grounds which are in the heart of the city and cover two square blocks. In 1906, he had removed to East Orange, N. J.,

* Memoir compiled from information supplied by E. S. Rice, Asst. Engr., A. T. & S. F. Ry. Co., Chicago, Ill., and on file at the Headquarters of the Society.

where he died on February 9, 1922. He is survived by his wife, four sons, and one daughter.

Mr. Dailey was a member of Lincoln Post No. 1, G. A. R., of Topeka, Kans. As a Thirty-second Degree Mason, he was a member of Siloam Lodge No. 225, of Topeka, and he was also a member of the First Unitarian Church of Orange, N. J.

Mr. Dailey was elected an Affiliate of the American Society of Civil Engineers on September 7, 1904.

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